KING COUNTY CONVEYANCE SYSTEM IMPROVEMENT PROJECT

JUANITA BAY PUMP STATION

TASK 310 REPORT

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KING COUNTY
Department of Natural Resources

ACKNOWLEDGEMENTS

The following people contributed to the writing of this report:

King County Department of Natural Resources
Wastewater Treatment Division

Bob Peterson (Conveyance System Improvements Project Manager)
Roger Browne
Ed Cox
David Dittmar
Ron Kohler
Katherine McKee
Darrell Myers
Chris Okuda
Bob Swarner (Flow Projections)
Mark Lampard (Flow Projections)

HDR Engineering, Inc.

James Peterson (Project Manager)
Samuel Perry

Table of Contents

Page

EXECUTIVE SUMMARY **E-1**

CHAPTER 1 – INTRODUCTION **1-1**

CHAPTER 2 – FLOW PROJECTIONS **2-1**

Flow Projections 2-1

Potential for I/I Reduction 2-1

CHAPTER 3 – EXISTING JUANITA BAY PUMP STATION **3-1**

Raw Sewage Pumping System 3-3
- Pump Station Capacity 3-5
- Control System 3-6
- Primary and Standby Power System 3-9
- Water System 3-10
- Air System 3-11
- Drainage System 3-11
- HVAC System 3-11
- Equipment Accessibility and Maintenance Considerations 3-12
- Personnel Egress, Safety, and Code Issues 3-13
- Flow Isolation and Bypassing Capabilities 3-14

Juanita Forcemains 3-14
- Hydraulic Transients 3-17
- Additional Forcemain Concerns 3-18

Conclusions 3-19

CHAPTER 4 – ALTERNATIVE EVALUATION **4-1**

Pump Station Design Flows 4-1

Design Standards and Code Requirements 4-1

Site Constraints 4-3
- Geotechnical Issues 4-3

Pump Station Alternatives 4-4

Alternative 1 – New Pump Station on Existing Site 4-4
- Facility Description 4-5
- Construction 4-5

Alternative 2 – New Pump Station on a Nearby Site 4-6
- Facility Description 4-7
- Construction 4-7

Alternative 3 – Construct A New Juanita Heights Pump Station.................................. 4-9
Alternative 4 – Upgrade of Existing Pump Station and Storage 4-10
Pump Station Upgrade.. 4-10
Alternatives for Increasing Pump Station Capacity.. 4-11
Limitations of Upgrade.. 4-14
Upstream Storage.. 4-15
Construction... 4-17
Cost Estimates.. 4-17

CHAPTER 5 – CONCLUSIONS AND RECOMMENDATIONS..............................5-1
Comparison of Alternatives 5-1
Recommended Project 5-3
Appendix A – Planning Level Drawings
Appendix B – Cost Estimates

LIST OF FIGURES

Figure **Page**

Figure 1-1 – Existing Pump Station and Forcemains 1-2
Figure 3-1 – Existing Pump Station.. 3-2
Figure 3-2 – 18-Inch Forcemain Pump Curves.. 3-7
Figure 3-3 – 24-Inch Forcemain Pump Curves.. 3-8
Figure 3-4 – Emergency Generator.. 3-9
Figure 3-5 – Raw Sewage Pump Number 104.. 3-13
Figure 3-6 – Forcemain Profile.. 3-15
Figure 4-1 – Pump Station Alternative Sites 4-8
Figure 4-2 – Juanita Bay Pump Station Peak Flow Hydrograph and Storage Estimate... 4-16

LIST OF TABLES

Table **Page**

Table 1-1 – Construction Information 1-1
Table 2-1 – Flow Projections 2-1
Table 2-2 – Reduced I/I Flow Projections 2-2
Table 3-1 – Pump Parameters 3-3
Table 3-2 – Pump Suction Velocities 3-4
Table 3-3 – Horizontal Velocities In The Wetwell 3-4
Table 3-4 – Hydraulic Detention Times 3-5
Table 3-5 – Pump Station Capacities 3-6
Table 3-6 – Forcemain Velocities 3-16
Table 3-7 – Forcemain Valves And Associated Structures 3-16
Table 3-8 – Summary Of System Problems 3-20
Table 4-1 – Influent Sewer Storage Volume 4-5
Table 4-2 – Horizontal Velocities in the Wetwell 4-12
Table 4-3 – Pump Section Velocities 4-12
Table 4-4 – Pump Discharge Line Velocities 4-13
Table 4-5 – Pump Discharge Header Velocities 4-13
Table 4-6 – Summary of Design Criteria Exceeded 4-14
Table 4-7 – Estimated Pump Station Capital Costs (1999 Dollars) 4-18
Table 5-1 – Comparison Of Alternatives 5-2
Table 5-2 – Preliminary Construction Schedule 5-4

EXECUTIVE SUMMARY

The Conveyance System Improvement (CSI) Project involves planning efforts on an array of pump station and conveyance system improvements. The purpose of this report is to evaluate the existing Juanita Bay Pump Station and related conveyance system components, and outline alternatives to convey the projected increased flows out of the basin.

The Juanita Bay Pump Station service area is approximately 7,650 acres, including 7,000 acres in the Northshore Utility District, and 650 acres in the City of Kirkland. The 20-year peak flow to the Juanita Bay Pump Station is projected to significantly increase over the next 50 years as the basin is further developed, and inflow and infiltration (I/I) increases as the collection system deteriorates with age.

The Juanita Bay Pump Station has a number of operational problems under existing flow conditions that will be exacerbated as flows increase. These problems include:

- Horizontal velocities in the wetwell are higher than recommended at peak flows and surface vortexes have been observed in the wetwell under existing conditions.
- The velocities in the pump suction draft tubes are significantly higher than recommended. This suction piping cannot be replaced without removal of all the equipment and piping in the bottom floor of the pump station.
- All of the electrical equipment including the transformer, motor control centers, and generator are too small to handle the increased electrical load required to convey higher flows.

The estimated peak capacity of the existing Juanita Bay Pump Station, 14.2 mgd, is well below the projected 20-year peak flow of 19.9 mgd for the year 2000. Based on these flow projections and constraints associated with the existing pump station site, five alternatives were evaluated to either upgrade or replace the existing pump station. The recommended project consists of a new 24 mgd peak capacity pump station near the northeast corner of 93rd Avenue NE and Juanita Drive NE. The pump station should be expandable to a peak capacity of 28 mgd, the projected 20-year peak flow without I/I reduction, should an I/I control program fail to compensate for the projected deterioration of the collection system.

This recommended project was selected since it would cost less than most other alternatives, would not require exemptions from zoning codes, would have fewer construction impacts and constraints than other alternatives, and would result in a pump station that meets the latest County design standards, unlike the existing Juanita Bay Pump Station.

Unless I/I control is implemented for the basin or deterioration of the collection system is not as great as projected the 28-mgd peak flow would require construction of additional force main by 2016.

CHAPTER 1 - INTRODUCTION

The Conveyance System Improvement (CSI) Project involves planning efforts on an array of pump station and conveyance system improvements. The purpose of this report is to evaluate the existing Juanita Bay Pump Station and related conveyance system components, and outline alternatives to convey the projected increased flows out of the basin.

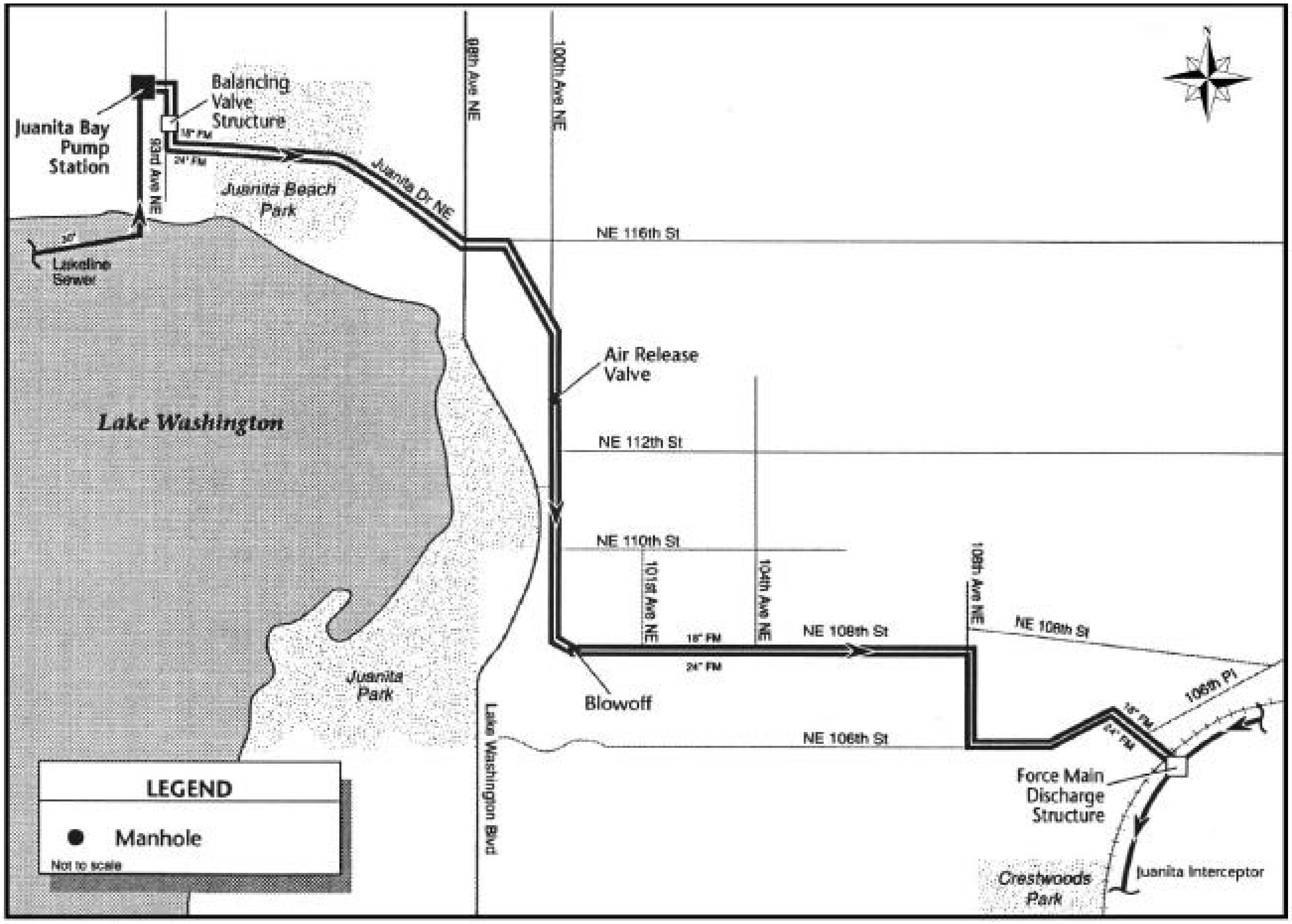
The Juanita Bay Pump Station is located on the northwest corner of the intersection of 93rd Avenue N.E. and N.E. Juanita Drive in the City of Kirkland. Construction of the original pump station was completed in 1969 (Table 1-1). In 1970, the pump station superstructure was added, which included the emergency generator, bathroom, equipment room, and the motor control centers (MCCs). The Juanita Bay Pump Station previously discharged to the Juanita Heights Pump Station. Under these conditions, the static head on the pump was approximately 45 feet.

Table 1-1. Construction Information

Construction	Date Complete
Constructed Juanita Bay and Juanita Heights Pump Stations. Constructed 18-inch force main.	1/69
Installed standby generator. Constructed superstructure. Modified Juanita Bay and Juanita Heights Pump Station control systems.	6/70
Abandoned Juanita Heights Pump Station. Upgraded Juanita Bay Pump Station with new pumps, motors, variable frequency drives, and programmable controller. Constructed 24-inch force main	1/85

In 1985, the Juanita Heights Pump Station was abandoned. As a result, the static head on the pumps in the Juanita Bay Pump Station increased to 163 feet requiring the replacement of the existing pumps. The most recent upgrade also included construction of an addition to the superstructure to house variable frequency drives (VFDs) and other electrical and control equipment. As a part of the upgrade, a 24-inch force main was constructed and portions of the existing 18-inch force main were abandoned and new sections constructed parallel to the new 24-inch force main. Since the time of the last upgrade, there have been some minor modifications to the pump station including replacement of a pump that failed and replacement of the sump pump.

The Juanita Bay Pump Station discharges into the Juanita Bay Pump Station Force Mains, which are composed of parallel 18-inch and 24-inch force mains that extend approximately 9,680 lineal feet from the pump station to the Juanita Interceptor (Figure 1-1). The East Side Interceptor, in turn, conveys wastewater to the South Treatment Plant in Renton.



KING COUNTY
Conveyance System Improvements
Juanita Bay Pump Station

Figure 1-1
Existing Pump Station and Forcemains

CHAPTER 2 – FLOW PROJECTIONS

The Juanita Bay Pump Station service area is approximately 7,650 acres, including 7,000 acres in the Northshore Utility District, and 650 acres in the City of Kirkland. Currently, less than 85 percent of the basin is sewered. Peak flows to the Juanita Bay Pump Station are projected to almost double over the next 50 years as the basin is further developed and multistory residential units replace single family homes.

FLOW PROJECTIONS

King County's May 1999 flow projections indicate that the 20-year peak flow to the pump station will increase to 28 mgd in 2050 (Table 2-1). As the basin is further developed, the 1990 base flow of 3.3 mgd is projected to more than double to 7.4 mgd in 2050. In addition, the infiltration and inflow (I/I) flows were projected to increase because of development and deterioration of the collection system. The 20-year peak flows include a 7 percent increase in I/I rates per decade up to a maximum of almost 29 percent over 1990 conditions.

Table 2-1. Flow Projections

Year	Sewered Area (acres)	Base Flow (mgd)	5-yr I/I (gpad)	5-yr peak (mgd)	20-yr I/I (gpad)	20-yr peak (mgd)
1990	6,277	3.3	1,840	14.9	2,280	17.6
2000	6,414	4.3	1,970	16.9	2,440	19.9
2010	6,720	4.9	2,110	19.0	2,610	22.4
2020	7,027	5.6	2,240	21.4	2,770	25.1
2030	7,027	6.2	2,380	22.9	2,940	26.8
2050	7,027	7.4	2,380	24.1	2,940	28.0

POTENTIAL FOR I/I REDUCTION

King County is currently developing an I/I reduction program. If the I/I program can offset the effect of deterioration of the collection system, the peak flow projections could be revised downward. As shown in Table 2-2, the peak 20-year storm flow in 2050 with an I/I reduction program would be 23.4 mgd versus 28.0 mgd, a reduction of 4.6 mgd. This reduction in I/I may alleviate the need for or allow for the delay of downstream capital improvement projects such as paralleling the Juanita Bay Pump Station force mains or sections of the Eastside Interceptor (ESI). Where applicable, potential capital cost savings associated with I/I reduction in the Juanita Bay Pump Station basin will be discussed in association with the analysis of the alternatives to convey the projected flows.

Table 2-2. Flow Projections with an I/I Reduction Program

Year	5-yr peak (mgd)	20-yr peak (mgd)
1990	14.9	17.6
2000	16.1	18.9
2010	17.3	20.2
2020	18.5	21.6
2030	19.1	22.2
2050	20.3	23.4

CHAPTER 3 – EXISTING JUANITA BAY PUMP STATION

The Juanita Bay Pump Station is a four level structure, three levels of which are below grade (Figure 3-1). The rectangular above grade structure contains a control room, equipment room, generator room, restroom, and wetwell access. The control room, generator room, and wetwell access each have separate outside entrances. The control room houses four variable frequency drives (VFDs), the main control panel, automatic transfer switch, main circuit breaker, and other electrical cabinets. The air compressors and associated level control equipment for the bubbler system are located adjacent to the control room in the equipment room. The generator room occupies the majority of the floor space of the upper level and houses approximately half of the electrical and control cabinetry for the pump station as well as a 400 kW generator sized to operate two pumps.

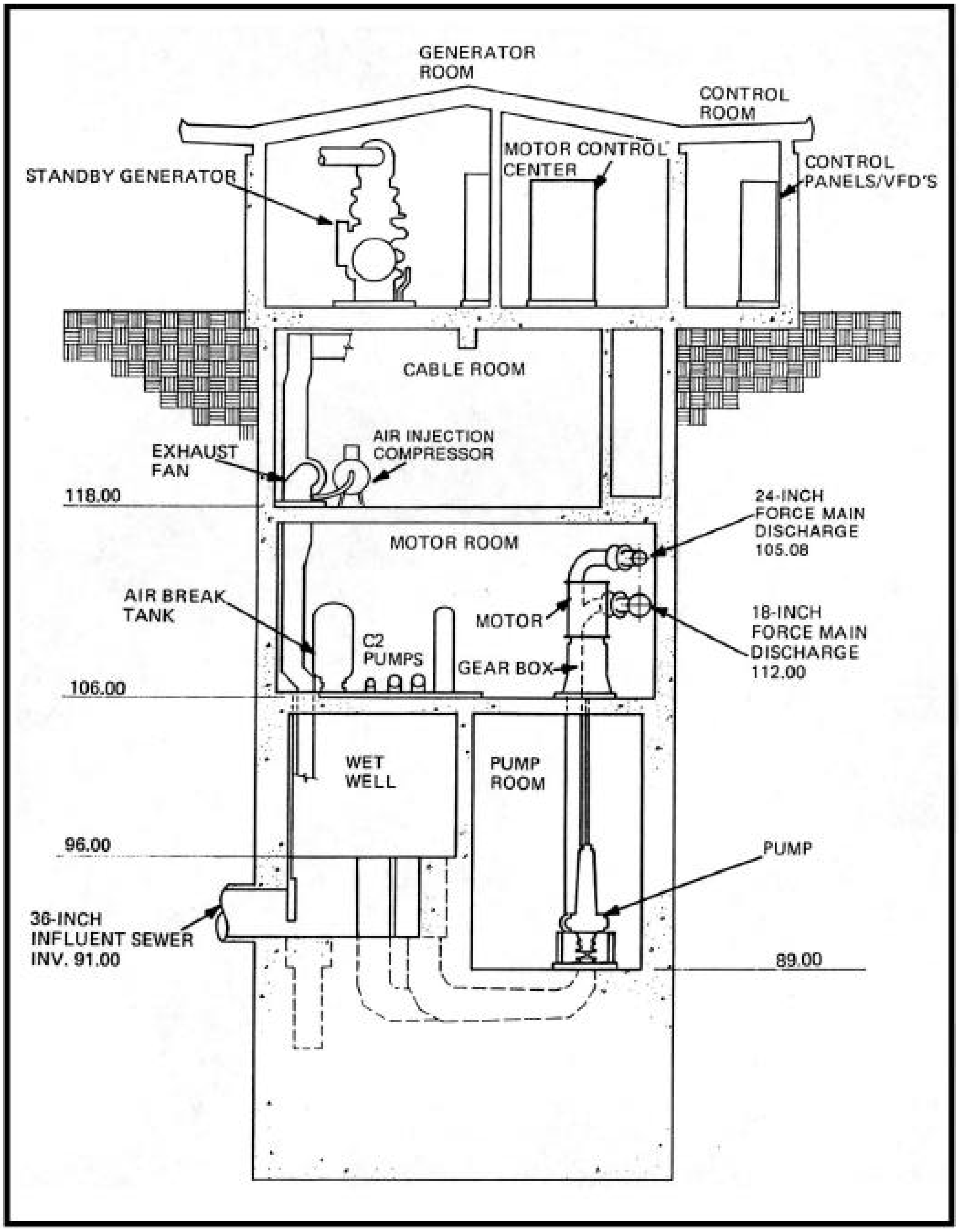
The lower three levels of the pump station consist of a 26-foot outside diameter caisson, the bottom of which is approximately 54 feet below grade. The thickness of the caisson walls taper from 2 feet on the bottom level to 1 foot on the first level below grade.

The bottom level consists of the pump room and wetwell. The pump suction extends from the wetwell into the floor slab beneath the pump room. Each pump suction draft tube tapers from 2-foot inside diameter at the bottom of the wetwell to 12-inch inside diameter at the knife gate isolation valve beneath each pump. Sewage is discharged from each pump via 12-inch lines to 12-inch and 18-inch discharge headers in the motor room on the second level.

The cable room above the motor room houses most of the HVAC equipment for the unclassified areas in the pump station as well as a utility sink. The force mains penetrate the floor of the cable room, increase in size to 18 and 24 inches respectively, turn, and penetrate the caisson. Air vacuum valves and surge relief valves are located on each force main to minimize the damage caused by surges resulting from power failures and equipment malfunctions.

Seal water manifolds are on the wall behind Pump 104. Also located in the pump room near Pump 104 is the sump drainage system. This system includes one sump pump that discharges into a 3-inch force main that conveys pumped drainage to the upstream manhole.

The wetwell access room houses the air break tank, the water system backflow preventer. A spiral staircase provides access to the wetwell from the wetwell access room.



KING COUNTY
Conveyance System Improvements
Juanita Bay Pump Station

Figure 3-1
Existing Pump Station

The motor room houses the raw sewage pump motors and the non-potable (C2) water system. The C2 water system includes an air break tank, hydropneumatic tank, and three pumps, one for flushing water, and the other two for the high pressure C2 water system.

RAW SEWAGE PUMPING SYSTEM

The raw sewage pumping system includes four variable speed pumps (Table 3-2). There is a manually operated sluice gate on the influent sewer. The coarse bar screen upstream of the pump suction lines that was originally installed has been removed. The pumps normally operate automatically in response to changes in the elevation of wastewater in the wetwell. Typically, the pumps operate with wastewater elevations in the wetwell between 93.0 and 95.8 feet. However, during storm events, the wetwell is surcharged to increase the net positive suction head available (NPSHA) to minimize problems with cavitation at the pumps, a reported problem at the pump station. This surcharging has damaged the piping and other equipment located in the wetwell. Some deterioration of the wetwell walls has also occurred.

Table 3-1. Pump Parameters

Type	Screw Centrifugal
Design Flow	3,200 gpm
Design Discharge Pressure	200 feet
Speed	1,350 RPM
NPSH Required at Design Conditions	18 feet
Motor Horsepower	250 HP

There are several problems with the ability of the wetwell to convey existing flows and any future increase in flows. Vortexing is evident at lower flows, and velocities in the wetwell are excessively high during peak flows. The pump suction draft tubes taper from an inside diameter of two feet at the bottom of the wetwell to 12 inches just upstream of the pump inlet. The velocities in this suction line exceed 5.0 feet-per-second (fps), a practical maximum velocity, for both existing and future flows (Table 3-3).

Horizontal velocities in the wetwell are also higher than recommended at peak flows (Table 3-4). These high horizontal velocities likely contribute to prerotation of the wastewater entering the draft tubes leading to vortexing. The pump station was designed with vortex suppressor plates in the draft tube inlets. However, prior to their removal, these vortex suppressors may themselves have helped to create vortexes.[1] As flows to the pump station increase, the severity of vortexing will likely increase resulting in cavitation, increased wear

[1] Recommended design includes holes in the vortex suppressors. "The holes in (the vortex suppressor) must be adequate for distributing any eddies shed at the free edge of the plate by cross flow to prevent the plate itself from becoming a vortex generator" (Ref. Pumping Station Design, Sanks et al, pg. 310).

on the pumps, and decreased pump efficiency and capacity. In addition, the flow distribution to the pumps is reported to be unequal with higher flow going to the center two pumps in the wetwell.

Table 3-2. Pump Suction Velocities

$Q_{pump\ station}$ [1] (mgd)	$V_{draft\ tube\ inlet}$ (fps)	$V_{pump\ inlet}$ (fps)	Notes
10.2		*5.0*	Maximum recommended velocity in pump suction.[2]
18.4	2.2	**9.0**	Rated capacity of existing pumps.
28.0	3.5	**18.4**	Future pump discharge based on 28-mgd peak pumping capacity for the station.
28.3	*3.5*		Maximum recommended velocity in draft tube inlet.[2]

Notes:
1. Based on all four pumps operating and equal flow to each pump.
2. Reference: Pump Station Design, Sanks et al, 1st Ed. (1989). Pg. 312.
3. Velocities in excess of recommended parameters are **highlighted**.

Table 3-3. Horizontal Velocities in the Wetwell

$Q_{pump\ station}$ (mgd)	V_{horiz} [1] (fps)	Notes
4.7	0.7	Average daily flow (1990).[3]
7.1	1.0	Maximum recommended velocity.[2]
10.6	**1.5**	Peak flow (1990).[3]
13.4	**1.9**	Nominal firm pumping capacity.[3]
28.0	**4.0**	Peak 20-year flow (2050) without I/I reduction.

Notes:
1. Reference: Offsite Facilities and Miscellaneous Structures Manual. Updated 12/1994.
2. Reference: Pump Station Design, Sanks et al, 1st Ed. (1989). Pg. 312.
3. Horizontal velocity calculated in channel just upstream of the inlet to the first pump.
4. Velocities in excess of the recommended value are **highlighted**.

The speeds and resulting pumping capacities of the all four pumps are controlled by variable frequency drives (VFDs) located in the control room. This equipment is undersized for the larger motors that will be required to convey higher flows. Currently, ambient air is used to provide cooling for the control room. Air is drawn into the control room from the generator room and exhausted through the roof. To minimize the potential for overheating the VFDs and the resulting pump failure of a pump, direct mechanical cooling should be provided for VFDs as a part of the pump station upgrade. The County is currently evaluating replacement of the three oldest VFDs. These VFDs were installed as part of the 1985 pump station upgrade while the VFD for Pump No. 1 was installed in 1994.

Four new VFDs should be included as part of any pump station upgrade. In accordance with the new King County standards under development, the new VFDs should include 12 or 18 pulse drives to minimize the harmonics and improve motor life. While new inverter duty rated motors are adequate for operation with VFDs, any reduction in insulation stress and motor heating will prolong life and reduce chances of motor failure.

In the event of a mechanical or power failure to the pump station, the wetwell will be surcharged and wastewater stored in the Holmes Point Trunk. The Holmes Point Trunk is a 30-inch diameter line that extends approximately 8,400 feet from the wetwell to the Holmes Point Flushing Structure. In the event of a catastrophic failure of the pump station, wastewater would be stored in this trunk line from 18 minutes to over 1.5 hours depending upon the flow to the pump station (Table 3-5). Relative to the storage volume in the Holmes Point Trunk, the storage volume in the wetwell is negligible even under surcharged conditions. Available storage times will be slightly higher than that indicated since there will be some storage in other sewers upstream of the pump station.

Table 3-4. Hydraulic Detention Times

Q (mgd)	Hydraulic Detention Time[1] (min)	Notes
4.7	95	Average daily flow (1990).[2]
10.6	42	Peak flow (1990).[2]
13.4	33	Nominal firm pumping capacity.[2]
23.4	18	Peak 20-year flow at (2050) with I/I reduction.
28.0	15	Peak 20-year flow at (2050) without I/I reduction.

Notes:
1. Calculated hydraulic detention time assumed entire flow to the pump station would be stored in the Holmes Point Trunk and storage in other influent lines is negligible.
2. Reference: Offsite Facilities and Miscellaneous Structures Manual. Updated 12/1994.

Based on this analysis of the existing raw sewage pumping system, it appears that increasing flows to the pump station will exacerbate already significant problems associated with vortexing and high pump suction velocities. Surcharging the wetwell, as is currently done during high flow periods, will reduce horizontal velocities in the wetwell. More importantly, surcharging provides additional net positive suction head (NPSH) thereby reducing the potential for cavitation in the suction line and resulting poor pump performance. However, surcharging alone will not mitigate the other significant problems associated with the wetwell. The draft tubes are cast into the floor beneath the wetwell and pump room. This suction piping cannot be replaced without removal of all the equipment and piping in the bottom floor of the pump station. Removal of the suction piping would be difficult if not impossible without temporarily bypassing flow around the pump station. Therefore, the alternatives to accommodate higher flows to the pump station should include an evaluation of replacement of the existing wetwell with one designed for higher flows and a provision to bypass the station during construction.

Pump Station Capacity

Normally, the pump station is operated so that each pump discharges into a dedicated force main. Since the hydraulic balancing valve was not installed between the two force mains, the headloss through the force mains will, in most cases, not be equal. Manufacturer pump curves were used to calculate system head curves for various operational conditions (Figures 3-3, 3-4). Firm, total, and standby pump capacities were determined from these system-head curves based on a Hazen-Williams Coefficient of 100 and a wetwell elevation of 94.0 feet (Metro Datum) (Table 3-6). When measured and calculated flows were compared, the values were typically within of 8 percent of each other. For a given wetwell elevation and pump RPM, the flowmeter readings varied by approximately 7 percent. Therefore, the calculated standby, firm, and peak capacities of 9.0, 10.8 and 14.2 mgd are reasonable and within the accuracy of the available data.

Table 3-5. Pump Station Capacities

	18-inch Force Main		24-inch Force Main		Total Flow
Total Pumps	Pumps	Flow (mgd)	Pumps	Flow (mgd)	(mgd)
1	1	4.0	0	0.0	4.0
1	0	0.0	1	5.0	5.0
2 (Standby Capacity)	1	4.0	1	5.0	9.0
2 (18-inch FM)	2	5.8	0	0.0	5.8
2 (24-inch FM)	0	0.0	2	8.4	8.4
3 (Firm Capacity)	2	5.8	1	5.0	10.8
3	1	4.0	2	8.4	12.4
4 (Peak Capacity)	2	5.8	2	8.4	14.2

Notes: Maximum recorded flow for the period 12/96 to 8/97 is 14.24 mgd (1/1/97 3:00 p.m.) This flow involved surcharging the wetwell to 108.27 feet, 12.52 feet above the “standby on” elevation of 95.75 feet.

Control System

The pump station is fully automatic, normally unattended, and can be controlled from panels in the pump station control room. Staff at the South Treatment Plant in Renton monitor the operations of the pump station through the Forney and Metrotel SCADA systems. These systems allow King County staff to monitor wetwell elevations, pump speed, and pump station flow, as well as be altered to the failure of critical process components. The programmable logic controllers (PLCs) are reportedly in need of replacement, but otherwise this system functions adequately for the existing station. The County is currently undertaking a program to replace existing Texas Instrument (TI) PM550 PLCs with Modicon units at offsite facilities.

FIGURE 3-2
18-INCH FORCEMAIN PUMP CURVES

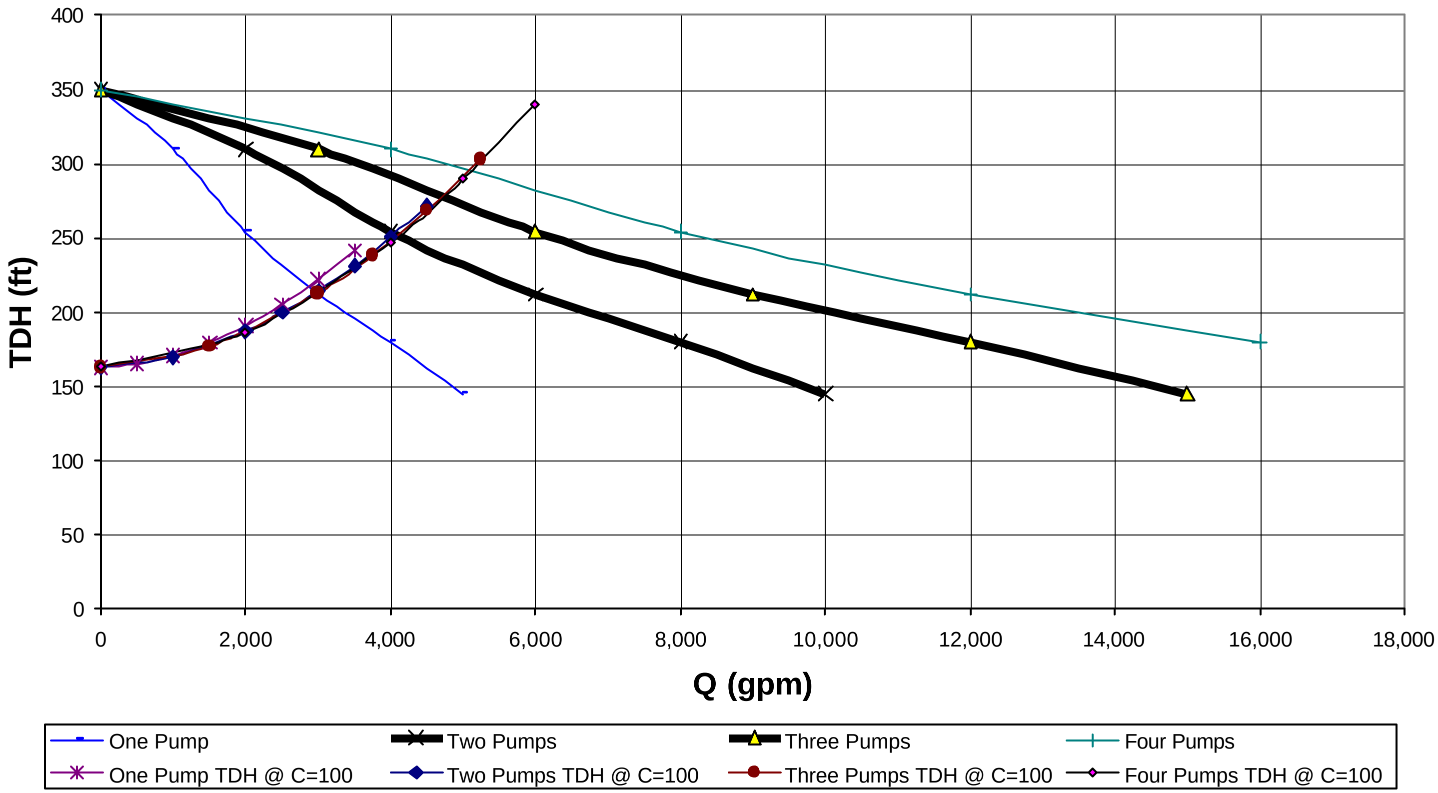
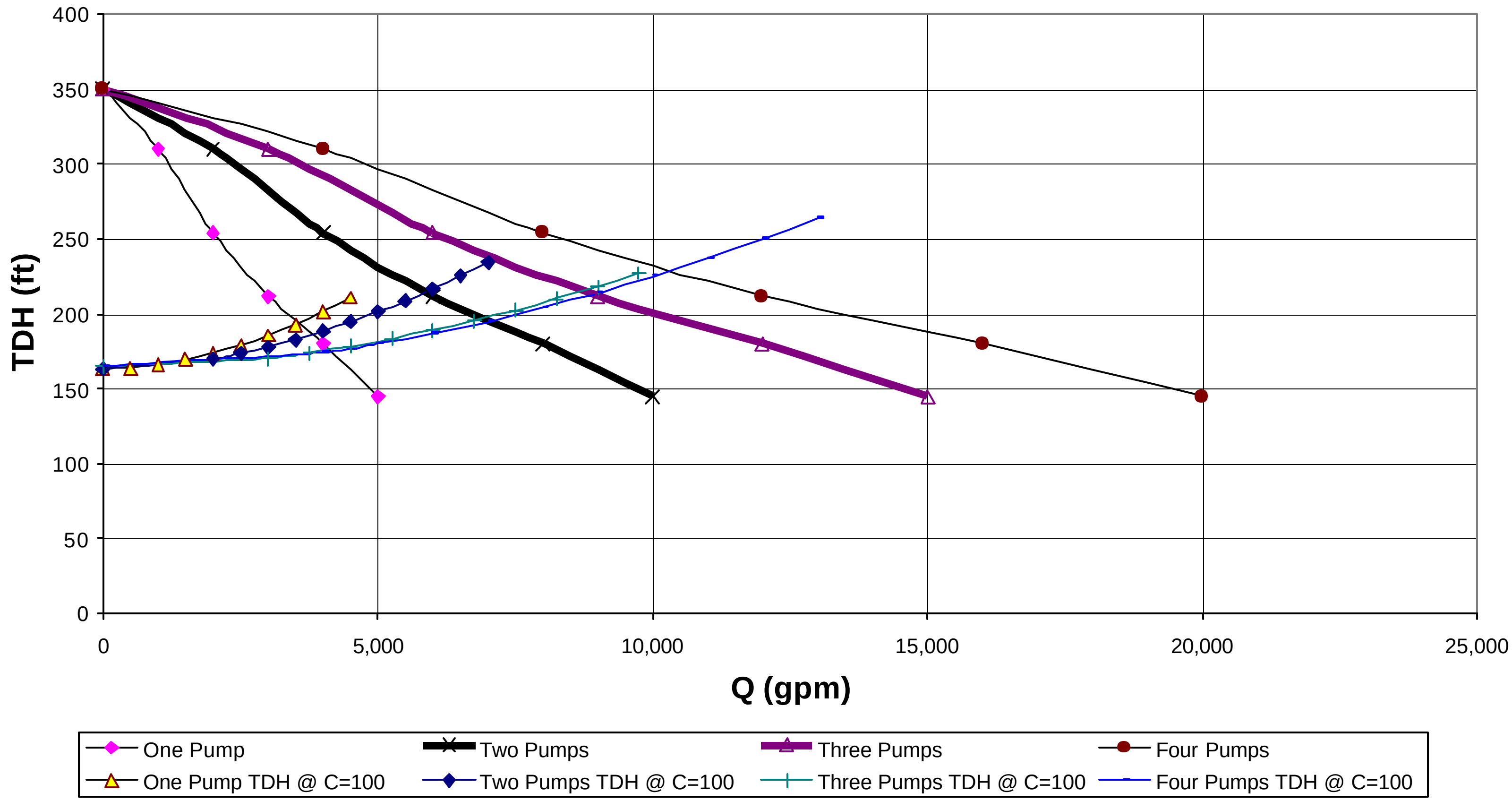


FIGURE 3-3
24-INCH FORCEMAIN PUMP CURVES



The wetwell includes a float switch for high level alarm, but currently lacks a redundant system of float switches to turn on the pumps should the bubbler system fail. If the existing wetwell is used to convey higher flows, float switches should be installed to provide a redundant means for controlling the pumps. The installation of a redundant raw sewage pump control system is King County standard practice for offsite facilities.

Primary and Standby Power System

The main power feed to the pump station is from the underground 500 kVA, 12,470 – 480Y/277V transformer located on the west side on the facility. This transformer is owned and maintained by Puget Sound Energy and located below grade in a vault. The transformer feeds the main 1200 amp bus in the control room that is adjacent to the Automatic Transfer Switch (ATS) for the generator.

In the event of a power failure, power is supplied to the station by a 500 kVA/400 kW generator located in the main equipment room (Figure 3-4). This generator has the capacity to operate two pumps, with a combined capacity of approximately 9 mgd. The peak flow recorded for 1990 was 10.6 mgd and the projected 20-year peak flow for the pump station in the year 2000 is 19.9 mgd. Therefore, there is a significant potential for an overflow to occur during storm flows accompanied by a power outage.

Figure 3-4: Emergency Generator

A single exhaust silencer and acoustical material on the ceiling provide limited sound attenuation for the generator. There is no noise attenuation on the louvers serving the generator. This minimal noise attenuation is a concern for meeting internal and property line noise requirements. According to state law adopted by the City of Kirkland (WAC 173-60-040), the maximum permissible daytime noise level for residentially zoned lots at the property line is 55 dBA and 45 dBA is the maximum permitted nighttime noise level. Exceptions are made for emergency equipment for the health, safety, or welfare of the community, such as a pump station generator. Improved noise attenuation for the generator is planned and scoping for the final design currently is in progress.

The existing diesel fuel storage tank is in compliance with the latest regulations for underground storage tanks and certified as such by the Washington State Department of Ecology (WSDOE). The capacity of the tank is approximately 900 gallons. Based on a full-load fuel consumption rate of 30 gallons per hour, the fuel tank has sufficient storage capacity to run the generator for 30 hours. A minimum of 24 hours of storage is typically provided for County offsite facilities. The existing storage tank meets this requirement. However, this storage tank will not be able to meet the 24-hour storage requirement if the generator is replaced with a larger unit.

All of the electrical equipment including the transformer, motor control centers, generator, and conduit are too small to handle the increased electrical load required to convey higher flows and will need to be replaced. A preliminary estimate of the required new electrical equipment includes:

- A 2,000 kVA pad mounted transformer,
- A 1,500 kW generator and fuel tank, and
- New electrical switchgear, service line, and MCCs.

Water System

The existing water system includes both potable (C1) and non-potable (C2) water systems. There is a 1-1/4 inch connection that serves both systems. The C1 water system provides water for drinking, the restroom, and landscape irrigation. The irrigation system has two vacuum breakers for backflow prevention. The C2 water system is separated from the C1 water system by an air break tank and subsequently is divided into two subsystems, one for flushing water and the other primarily for seal water for the raw sewage pumps. The C2 low pressure water system provides water for station washdown, pipe and pump flushing, and general maintenance. The C2 high pressure (C2HP) water system includes two high head (200 feet TDH) pumps to serve the pump seals. There have been no reported failures of this system and it appears to meet code.

Each of the two C2HP pumps is rated for 3.5 gpm at 200 feet. Two pumps are necessary to ensure seal water is provided in case one pump fails. Good design practice includes providing 2 gpm of seal water for each pump at 5 psi above the pump discharge pressure. Since the discharge pressure on the pumps will exceed 200 feet when two pumps are discharging to a force main, the existing seal water system appears to be undersized. The seal water pumps need to be replaced to prevent contaminants and abrasive material from entering the packing and causing excessive shaft wear.

The County primarily uses stuffing boxes with packing to seal against raw sewage leakage around the pump shaft. Historically, the County has not had good success with mechanical seal systems once the seals were replaced or rebuilt. For these reasons, the County has used mechanical seals, including flushless mechanical seals that do not require seal water, in only a very limited number of applications. A reliable seal water system is essential for continued successful operation of the main sewage pumps.

Air System

The air system provides starting air for the emergency generator, instrument air for the wetwell bubblers, air break tank, sump bubbler, and other miscellaneous equipment. The system consists of two completely redundant systems including redundant air compressors, receivers, and ancillary equipment. The compressors alternate starts to improve the reliability of the system. This system has worked reliably, but is antiquated and should be replaced as a part of any pump station upgrade with equipment meeting the latest King County standards, which require reliable redundant systems.

Drainage System

The drainage system consists of sanitary, storm, and sump systems. The gravity sanitary subsystem collects wastewater from the restroom, air break tank drain, and motor room drains all of which flows into the wetwell. The storm drainage system collects wastewater from the roof drains, floor drains on the upper level of the pump station, and yard catch basins. This stormwater flows into a 6-inch storm drain southwest of the pump station. The sump system collects wastewater from the pump room floor drains, and various other drains. This wastewater flows into the sump located in the pump room behind Raw Sewage Pump No. 104 from which it is pumped via a 3-inch force main into the upstream manhole. The existing drainage system lacks a redundant sump pump and there is poor access to the existing pump. A redundant sump pump should be provided. However, a redundant sump pump will not fit within the existing sump, and thus would need to be located on the pump room floor where there is limited floor space.

HVAC System

The existing heating and ventilation system operates continuously to supply fresh air throughout the pump station. In addition, the system maintains a near constant temperature within the facility and removes noxious, odor producing gases from the wetwell. Air is supplied to the facility from two supply fans with a rated capacity of 5,490 cubic feet per minute (cfm).

The existing system is sufficient to supply the requisite amount of air for the facility. NFPA 820 requires that the drywell and other parts of the pump station not exposed to sewer gases be ventilated continuously at six air changes per hour (ACH). In addition, the system should be designed to maintain a net positive pressure in the drywell and connected areas. Both of these requirements are met by the existing system.

Air is supplied to the wetwell from the generator room. This approach for providing air to the wetwell was common when the pump station was built. However, this cross connection of air between the wetwell area and the pump/control rooms is prohibited by current codes unless all areas are classified as Class I, Division 2, Group C & D. This classification requires that all equipment be explosion proof, which currently it is not. Complete separation of wetwell and drywell air must be provided as a part of any pump station improvements.

Odor control is currently not provided for the pump station and reportedly there have been no odor complaints. As a result, there are no plans to provide odor control for the pump station wetwell in the near future. However, chemical injection to control odors and corrosion at the force main discharges has been proposed. The proposed chemical injection facilities would probably be located within the pump station.

Equipment Accessibility and Maintenance Considerations

There is limited access to some equipment in the existing pump station. For example, the seal water system for the four raw sewage pumps is located behind Raw Sewage Pump Number 104 requiring the operator to climb past high speed rotating equipment to adjust and maintain this equipment (Figure 3-5). The sump pump is located next to the seal water system and is similarly difficult to access.

There is a two-ton monorail and electric hoist in the equipment room to facilitate removal and installation of large pieces of equipment such as pumps and motors. This hoist is the only one in the pump station. There are eyebolts above the pumps and motors to allow these pieces of equipment to be lifted and moved into position so that the hoist can be attached. Additional hoisting equipment on each floor would improve the ease of heavy equipment removal, but is not necessary since the existing system, although cumbersome, allows this equipment removal.

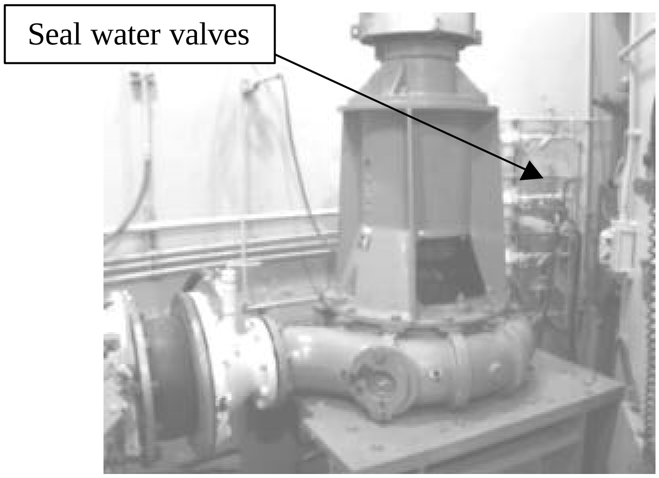


Figure 3-5: Raw Sewage Pump Number 104

Personnel Egress, Safety, and Code Issues

The Uniform Building Code (UBC) defines means of egress and fire protection requirements for structures. A pump station is normally classified as Group B occupancy, except when hazardous chemicals, such as caustic (NaOH) or sodium hypochlorite (NaOCl) are stored. These chemical storage areas are classified as Group H-7 occupancies if the quantity of these chemicals exceed exempt amounts. However, at this time, no hazardous chemicals are stored in the pump station in quantities that exceed exempt amounts. If caustic is used for force main slug dosing and odor control in the future, the storage areas will need to meet Group H-7 occupancy requirements.

In the Juanita Bay Pump Station, there are separate entrances for the equipment room and wetwell access. Two spiral staircases, one to access the wetwell and the other to access the drywell, provide means of egress from the lower levels of the pump station. Under current code, a single means of egress is sufficient as long as the footprint of all floors is less than 500 square feet and the occupant load is 10 or less. In addition, the maximum travel distance is 200 feet for nonsprinklered buildings such as the pump station. The maximum area for each floor below grade is significantly less than 500 square feet and the maximum travel distance is approximately 120 feet. Therefore, the pump station with a single means of egress from the pump room and another means of egress from the wetwell meets the requirements of the UBC.

Although spiral stairs are acceptable for certain limited situations described in the UBC and OSHA regulations, they are not recommend for new facilities when other means of egress can be provided[2,3]. However, the spiral stairs are already part of the pump station and

[2] Uniform Building Code (1997 – Latest Edition)- Section 1003.3.3 –"Stairs or ladders used only to attend equipment or window wells are exempt from the requirements of this section."

replacement of the spiral stairs in the existing pump station with another means of egress is not feasible given the space constraints of the existing facility.

Automatic sprinklers systems are required in the two following situations for the existing pump station:

- For basements and stories exceeding 1,500 square feet without requisite exit openings; and
- For Group H occupancies.

Hazardous chemicals currently are not stored in the pump station and the floors of the pump station are all less than 1,500 square feet so automatic sprinklers are not required. However, if hazardous chemicals are stored in the pump station for odor control in quantities that exceed exempt amounts, the space would be rated with Group H-7 occupancy and sprinklers would be required. This issue must be examined in more detail at the time of predesign to ensure compliance with the applicable sections of the UBC and UFC.

Flow Isolation and Bypassing Capabilities

In case of an extreme emergency, flow can be diverted from the pump station by closing the manually operated 18-inch by 30-inch influent sluice gate. This closure will cause an overflow at the Holmes Point overflow structure. Pumps can be individually isolated by closing the electric actuated knife gate valve on the bottom of each pump and closing the manually operated plug valves from each pump to the two discharge headers in the motor room. There have been pump failures that have necessitated isolating a pump. Reportedly, there was some difficulty in closing the suction isolation valve. This difficulty may have been due to the fact that the packing for the unbonneted knife gate valves may have been tightened so firmly to prevent leakage that closure of the valve became difficult.

JUANITA FORCE MAINS

Wastewater is discharged from the Juanita Bay Pump Station into two parallel force mains, one 18-inch and the other a 24-inch, which extend approximately 9,680 feet from the pump station to the discharge point in the Juanita Interceptor (Figure 3-9). If the headloss is equal between the two force mains, the 24-inch diameter force main will convey approximately 68 percent of the flow while the 18-inch line will convey the remaining 32 percent. Based on this assumption, the velocity in the 20-inch force main will increase to 7.5 fps in 2010 (Table 3-7). A practical maximum velocity is 8 fps. This velocity will be exceeded

[3] OSHA 1910.24 (b) – " . . .Spiral stairs shall not be permitted except for special limited usage and secondary access situations where it is not practical to provide a conventional stairway."

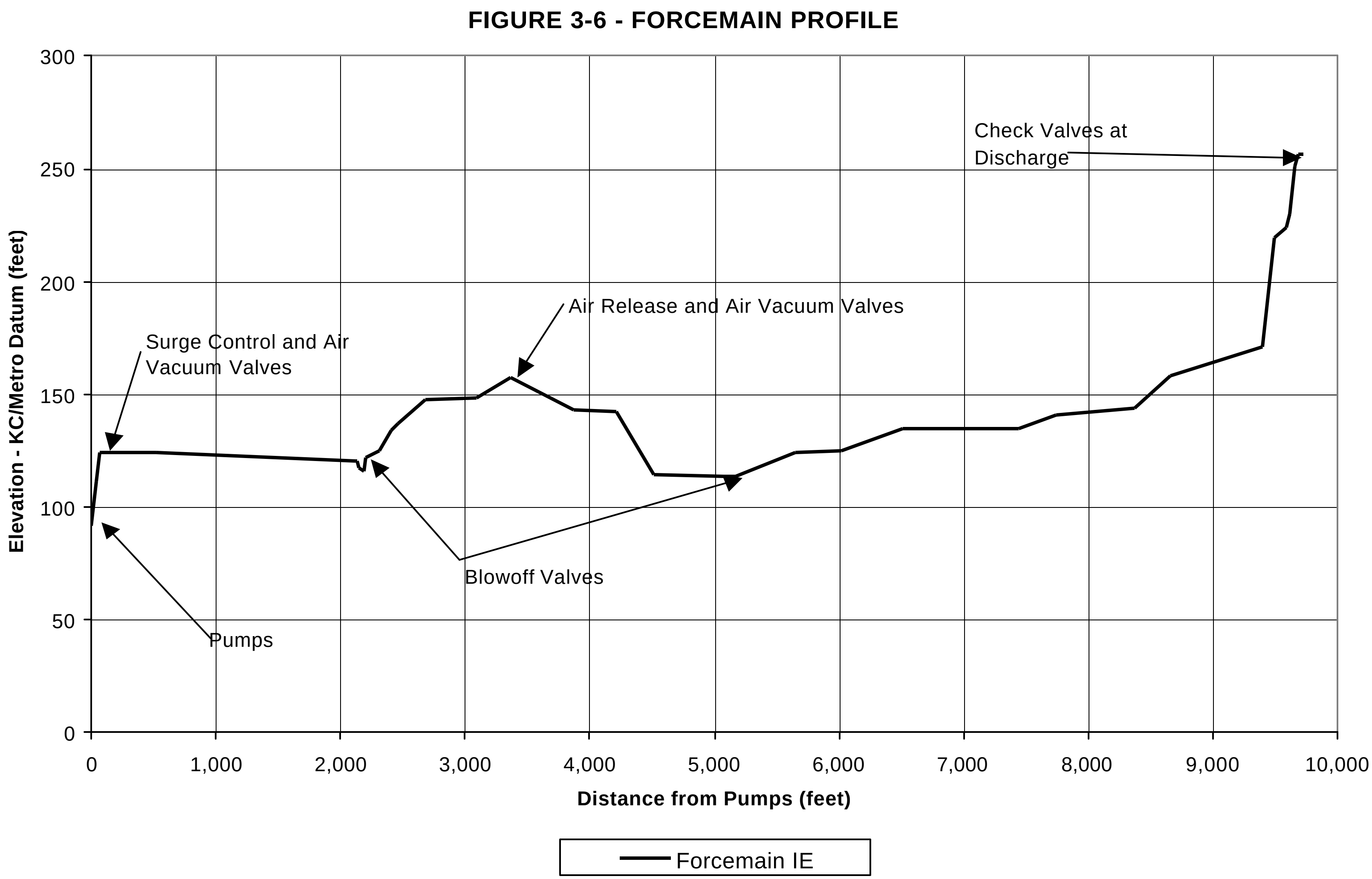


FIGURE 3-6 - FORCEMAIN PROFILE
Elevation - KC/Metro Datum (feet)
300
250
200
150
100
50
0
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
Distance from Pumps (feet)
Check Valves at Discharge
Air Release and Air Vacuum Valves
Surge Control and Air Vacuum Valves
Blowoff Valves
Pumps
Forcemain IE

by 2020 if an I/I control program is not implemented. However, if an I/I control program us able to offset the effects of deterioration of the collection system, replacement of the force mains will not warranted for hydraulic reasons.

Table 3-6. Force Main Velocities

Flow		Force Main Velocity	
Criterion	**Rate (mgd)**	**18-inch (fps)**	**24-inch (fps)**
Peak Flow –2000	17.6	4.9	5.9
Peak Flow –2010	22.4	6.3	7.5
Peak Flow –2020	25.1	7.0	8.4
Peak Flow –2050	28.0	7.8	9.4

Blowoff valves are located at low points to allow sediment to be drained from the force mains while air release valves allow entrained air to be released from the force mains (Table 3-8). Air vacuum, check valves, and surge valves are also installed on the force mains to minimize the magnitude of pressure surges resulting from a valve closure or pump stoppage. These valves should be flushed and exercised periodically to minimize the accumulation of grease and scum, and prevent the isolation valves from freezing. Since grease and scum can significantly diminish the performance of the valves, these valves should be cleaned regularly. Check valves are installed in each force main just prior to the force main discharge into the Juanita Interceptor.

Table 3-7. Force Main Valves and Associated Structures

Valve Type	Diameter (In)	STA[1]	Force Main Invert Elevation [2] (Feet)	Location
Air/Vac Check	6	-	124.50	Pump station cable room
Surge Control	3	-	124.50	Pump station cable room
Blowoff	6	21+25	115.87	Intersection of NE 116th St. and 98th Ave. NE
Air Release	2	33+00	157.00	100th Ave NE near NE 113th St.
Air/Vac Check	3	33+00	157.00	100th Ave NE near NE 113th St.
Blowoff	8	50+18	113.48	Intersection of NE 108th St. and 100th Ave. NE
Air/Vac Check	2	96+80	256.65	Discharge to Juanita Interceptor
In-Line Check	18/24	96+80	256.65	Discharge to Juanita Interceptor

Notes:
(1) STA 0+00 defined as centerline of flex coupling approximately 40 feet from the exterior of the pump station caisson.
(2) Vertical datum is “Metro Datum” plus 100.00 feet.

Hydraulic Transients

The Juanita Bay Pump Station has experienced hydraulic transients sufficient to bend pipe supports within the station. In March 1999, Golden Anderson Industries a supplier of air release, air vacuum, and surge relief valves conducted a preliminary surge analysis for the force mains to estimate the potential severity of the hydraulic transients especially if flows to the pump station are increased. This surge analysis was performed for the 24-inch force main at a pump station discharge of 23.9 mgd, the previously documented peak flow at build-out[4]. The surge analysis was performed on the 24-inch force main because, assuming equalized headloss between the two force mains, the 24-inch will have higher velocities and more severe transients. These surge analysis parameters included:

- Initial Conditions: 11,420 gpm at 240 feet TDH.
- Estimated Total Combined Pump and Motor WR^2: 1400 ft-lbs.
- Surge Protection: Air/vac valves on the pump header and at the high point at STA 33+63.
- Pump Check Valves: Wafer with lever and spring closure (Golden Anderson Model 23 or equal)

The maximum upsurge values predicted by the model were compared to the following pressure threshold values of the existing force mains:

- Pump discharge piping and force mains: It was assumed that the force mains are primarily Class 50 ductile iron pipe, which has a rated working pressure of 250 psi plus a 100 psi surge allowance. Based on these assumptions, the maximum allowable upsurge is 350 psi (807 feet). Some sections of the force main are listed on the drawings as steel or concrete cylinder pipe. As noted on the drawings of the 18-inch force main, "Pipe B" concrete cylinder was rated for 300 psi working pressure. Similarly, "Pipe B" steel pipe was designed with a 3/8-inch wall. For this wall thickness and ASTM A36 pipe with a working stress of 18,000 psi, the rated working pressure of the pipe is 433 psi[5].
- Pump discharge and valve flanges: Valve and pump discharge flanges were assumed to be Class 125 which have a rated working pressure of 250 psi (577 feet).
- Thrust restraint: Thrust restraint blocks and harnesses were provided at and in proximity to bends in the force mains. For the 24-inch force main, the restraint harnesses in the

[4] Memo from Bob Swarner to Sarah Draper on the subject "Peak Flows and Force Main Capacity at the Juanita Bay Pump Station". July 24, 1997.

[5] All ductile iron and steel pipe is designed with a factor of safety of 2.

vicinity of the Juanita Bay Pump Station were designed for a thrust of 100 thousand pounds. This thrust corresponds to a pressure of 221 psi (510 feet).

An additional concern is that the section of 18-inch force main between the Juanita Bay Pump Station and the site of the former Juanita Heights Pump Station was not designed for the higher heads that resulted from the removal of the Juanita Heights Pump Station. A significant portion of this 18-inch force main was relocated concurrent with the construction of the 24-inch force main and abandonment of the Juanita Heights Pump Station. Incorporating the 18-inch force mains into the thrust blocks for the 24-inch force mains provided thrust restraint for sections of the 18-inch force main that were not removed. Therefore, it appears that the thrust restraint for the sections of force main constructed in 1969 was upgraded when the Juanita Heights Pump Station was abandoned.

The surge analysis was based on peak flow at build-out and indicated that the most severe hydraulic transients occurred approximately 30 seconds after a power failure. The maximum total upsurge pressure at this time was approximately 295 psi (680 feet) at the pump station, approximately 75 psi (173 feet) greater than the maximum design rating of the thrust restraint for the force main at the pump station, but less than surge rating for the force main. It was suspected that the air/vacuum valve was being slammed shut by the rejoinder of a water column separation. Replacement of the air/vacuum valves in the model with vacuum breaker valves reduced the pressure spikes to less than the initial pump discharge head of 104 psi (240 feet).

Based on this analysis, it appears that the existing surge protection is inadequate to control surge pressures to less than the rated thrust restraint of the force mains. Therefore, the air/vacuum valves should be replaced with vacuum breaker valves to cushion the rejoining water columns. This valve replacement should be sufficient to control the maximum upsurge to less than the rated pressure of the pipe, flanges, and thrust restraint. This preliminary hydraulic transient analysis should be confirmed with a more detailed, analysis during predesign.

Additional Force Main Concerns

A review of the construction files for the Juanita Bay Pump Station force mains was conducted by King County WTD personnel to identify the extent of additional investigations for force mains as part of the pump station predesign effort. As a result of this review, two high points were identified where internal corrosion of the force mains may have occurred due to the formation of air pockets. These high points were between the pump station and approximately STA 4+50, and in the vicinity of the air release vault at STA 33+00. Non-destructive testing of the force mains should be performed in these locations as part of the predesign effort to determine if replacement of these sections of the force mains is warranted.

CONCLUSIONS

The Juanita Bay Pump Station has a number of operational problems under existing flow conditions that will be exacerbated as flows are increased to the pump station. The items that will necessitate the reconstruction or replacement of the pump station include:

- Horizontal velocities in the wetwell are higher than recommended at peak flows. These high horizontal velocities likely contribute to prerotation of the wastewater entering the draft tubes potentially leading to vortexing. Surface vortexes have been observed in the wetwell under existing conditions.

- The velocities in the pump suction draft tubes are significantly higher than recommended. The draft tubes are cast into the floor beneath the wetwell and pump room. This suction piping cannot be replaced without removal of all the equipment and piping in the bottom floor of the pump station. Therefore, the alternatives to accommodate higher flows to the pump station should evaluate replacement of the existing wetwell with one designed for the higher flows predicted.

- All of the electrical equipment including the transformer, motor control centers, generator, and conduit are too small to handle the increased electrical load required to convey higher flows.

- Complete separation of wetwell and drywell air and improved wetwell ventilation needs to be provided.

The hydraulic capacity and potential hydraulic transients of the Juanita Bay Pump Station force mains were evaluated. Based on this evaluation, replacement of the force mains is not warranted for hydraulic reasons as long as an I/I control program is implemented to offset increased flows resulting from deterioration of the collection system. However, it appears that the existing surge protection is inadequate and the air/vacuum valves should be replaced with vacuum breaker valves to cushion the rejoining water columns. This valve replacement should be sufficient to control the maximum upsurge to less than the rated pressure of the pipe, flanges, and thrust restraint.

In addition, there are several problems with the ancillary systems for the pump station. Although these system do not necessitate major reconstruction of the pump station, there are problems which need to be addressed when considering alternatives for conveying flow from the Juanita Bay Pump Station to the East Side Interceptor (Table 3-9).

Table 3-8. Summary of System Problems

System	Significance: Lower	Significance: Higher	Notes
Juanita Bay Pump Station			
Wetwell		X	Severe vortexing, no storage, excessive horizontal and suction velocities.
Raw Sewage Pumps	X		Firm capacity of only 10.8 mgd.
Control System	X		No redundant system for controlling pumps should bubbler system fail.
Primary and Standby Power		X	Emergency generator can only run two pumps
Water		X	Seal water pumps do not have sufficient capacity.
Drainage	X		No redundant sump pump.
HVAC		X	Ventilation air for wetwell from drywell, a code violation.
Equipment Access	X		Poor access to some equipment, e.g., sump pumps and seal water system.
Personnel Egress and Safety	X		
Juanita Bay Force Mains			
Capacity	X		Velocities less than 8.0 feet per second if I/I control implemented to offset deterioration of the system.
Hydraulic Transients		X	Hydraulic transients severe enough to warp thrust restraint in motor room. Improved surge projection should be provided for the force main.

CHAPTER 4 – ALTERNATIVE EVALUATION

Based on the evaluation of the existing Juanita Bay Pump Station (JBPS) and force mains and flow projections, five alternatives were developed to convey projected peak flows from the JBPS Basin to the Juanita Interceptor. These alternatives range from offline storage and a major pump station upgrade to complete replacement of the existing pump station with a new pump station.

PUMP STATION DESIGN FLOWS

The pump station design flows were based on County requirements that the firm pumping capacity of the station be sufficient to convey the 5-year storm flow and the peak pumping capacity be sufficient to convey the 20-year storm flow. Based on the most recent flow projections, a pump station with peak capacity of 28 mgd and firm capacity of 24 mgd should be sufficient to accommodate projected flows in 2050. However, if an I/I control program is implemented that is sufficient to offset the projected deterioration of the collection system, the peak and firm pump station capacities would only need to be 23.4 mgd and 20.9 mgd respectively. In addition, the highest velocities in the force mains should be limited to 8.0 fps to avoid excessively high frictional losses. This peak velocity criterion will be met at a peak flow of 24 mgd assuming equal headloss between the two force mains. Flows greater than 24 mgd would require replacement of the 18-inch force main or a third parallel force main to convey the 20-year peak flows. Therefore, a peak pump station capacity of 24 mgd was used as a basis for the planning level alternatives developed in this report.

DESIGN STANDARDS AND CODE REQUIREMENTS

The conceptual conveyance alternatives were developed based on County pump station design requirements and applicable codes. These criteria include:

1. Wetwell Design: For the conceptual level alternatives, the following design practices were employed:
   - A downturned suction bell inlet with the bottom of the bell 0.33 bell diameters off the wetwell floor;
   - Velocity in the suction pipe of less than five feet-per-second;
   - Horizontal velocities in the wetwell of less than one foot-per-second.

2. Pump Speed Control: Variable frequency drives (VFDs) with 12 or 18 pulse units.

3. Standby Power: An emergency diesel generator sized to provide power to all the raw sewage pumps and other essential equipment including level and instrument control, HVAC equipment, sump pumps, water system pumps, and lighting. The standby power

system would include a diesel fuel storage tank with sufficient storage capacity to operate the generator for 24 hours at full load.

4. Chemical Handling Facilities: The December 1998 draft of the East Side Interceptor Chemical Injection Facility Feasibility Study recommended adding 395 gallons of 25-percent caustic (NaOH) twice a week to the pump station wetwell to control sulfide generation in the JBPS force mains. A 2,500-gallon storage tank would be sufficient to provide three weeks of caustic storage at the pump station at this slug-dosing rate. The area in which these chemicals are stored would be rated H-7 and all necessary code and safety requirements must be addressed for this part of any proposed pump station. Other chemicals evaluated, such as calcium nitrate, would require additional storage but eliminate most of the safety and permitting concerns associated with the use of caustic.

5. Equipment Access and Egress: Stairway treads at least 44 inches wide and at least 42 inches of working space around all major pieces of equipment to facilitate equipment maintenance and improve worker safety. Separate entrances are required for the wetwell, drywell, and chemical handling facilities.

6. HVAC: Complete separation of the wetwell and drywell HVAC systems. Forced ventilation with at least 12 air changes per hour for the wetwell and at least six air changes per hour for the drywell.

7. Sound Attenuation: To protect the hearing of operation and maintenance personnel and avoid the need for a noise monitoring program, the sound level in the pump stations should be maintained at less than 85 dBA. The generator was isolated from the rest of the pump station to minimize the noise transmitted from the generator to other parts of the facility. Sound emanating from the station will also need to meet the maximum permissible environmental noise levels (WAC 173-60). Given the residential neighborhood surrounding the existing pump stations, the applicable maximum continuous daytime noise level at the property line is 55 dBA and the maximum nighttime noise level is 45 dBA. It may be possible to obtain a variance for exercising the generator since it is emergency equipment necessary to protect the health of the community.

8. Wetwell Level Control: Wetwell bubblers including redundant air compressors and an air cylinder for backup supply. Backup floats for wetwell level control in the event the instrument air system fails.

9. Instrumentation: The instrumentation system will be based on using Modicon programmable logic controllers (PLCs), the current King County Standard with connections to the Forney and Metrotel SCADA systems for remote station monitoring and control.

10. Water Supply: Backflow prevention and air gap tank in accordance with the latest King County design standards.

11. Influent Sewer: The bottom seven inches of the existing 36-inch influent sewer was paved during construction of the pump station restricting its capacity to approximately

20 mgd without surcharging. Therefore, the existing interceptor should be replaced with at least a 36-inch line to convey the projected peak flows without surcharging. The pipe could be upsized to provide additional storage capacity.

12. Pig Launch: To clean the force mains as needed.

SITE CONSTRAINTS

The JBPS parcel is zoned Residential Multi-family (RM). The requirements for this zoning designation include 20-foot front, rear, and side yard setbacks and no more than 70 percent of the lot may be covered by a pump station or other structures. In addition, extensive landscaping is required including a 15-foot wide landscaped buffer and solid fence. It may be possible to obtain a variance from these requirements.

The existing pump station property is located approximately 470 feet north of the Lake Washington shore, significantly greater than the requisite 200-foot shoreline setback. In addition, the pump station is located approximately 320 feet east of the edge of Juanita Creek, a Class A stream. No surface modifications are allowed within 75 feet of Juanita Creek and no structures are may be constructed within 85 feet of the creek. Although this stream buffer does not impact the existing pump station site, the siting of new facilities and construction activities must account for this buffer.

Geotechnical Issues

The bottom of the JBPS caisson is 54 feet deep and the floor of the pump room is 41 feet below grade. Groundwater in the vicinity of the pump station is within a few feet of the ground surface. The caisson depth, high groundwater, and close proximity of adjacent structures limits the number of feasible construction techniques to modify the pump station structure. The 1966 soils investigation prescribed two primary alternatives for the construction of the JBPS. The following excerpt discusses these alternatives:

> “It is considered possible that the station could be constructed either with a cofferdam or as a caisson. The former requires dewatering if a tremie seal is not employed, bracing and sheeting or lagging. In view of the excavation depth, heavy lateral pressures on excavation supports are expected. Further, hard driving is expected for sheets or soldier beams to penetrate into the underlying firm soils. The latter requires a deeper excavation, approximately 54 feet including an 8-foot thick tremie slab. However, caisson construction would have the advantages of a supported excavation in an unrestricted environment. In view of the above, the

caisson method of construction appears to be more feasible and is therefore recommended.[6]"

Based on this information, the construction of a caisson was the first alternative evaluated when considering excavation for a new pump station. Where caisson construction was not considered feasible, other construction techniques were considered.

PUMP STATION ALTERNATIVES

As discussed in Chapter 3, the existing pump station has a number of problems that will only worsen as flows to the pump station increase. These include wetwell hydraulic problems and severe space constraints that make it impossible to upsize pumps and piping within the existing caisson to meet predicted peak flows. For these reasons, a number of alternatives were analyzed for conveying the design flows to the Juanita Interceptor. These alternatives included:

- A new pump station on the existing pump station site;
- A new pump station on an alternative site near the existing pump station site;
- Reconstructing the Juanita Heights Pump Station to reduce the pumping head thereby reducing the motor size and electrical loads required for the JBPS; and
- Upgrading the existing pump station to convey higher flows, a new generator and chemical feed building on a nearby site, and offline storage to attenuate peak flows.

ALTERNATIVE 1 – NEW PUMP STATION ON EXISTING SITE

The existing JBPS includes a 26-foot outside diameter caisson, the bottom of which is approximately 54 feet below grade. The existing superstructure contains the emergency generator, major electrical equipment, and some ancillary systems equipment. To utilize the existing site for a new pump station, the existing superstructure would need to be demolished. For this planning level design, it was assumed that the existing caisson would be gutted and converted into a wetwell. To convey wastewater during construction of the new pump station on the existing site, a temporary pump station would be required.

The JBPS parcel is zoned Residential Multi-family (RM). The requirements for this zoning designation include 20-foot front, rear, and side yard setbacks. It may be possible to obtain a variance from these requirements from the City of Kirkland. The existing pump station is

[6] Final Report Soils Investigation – Juanita Bay Pumping Station, Juanita Heights Pumping Station, Juanita Force Main – Kirkland, Washington. October, 1966.

within 15 feet of the street right-of-way and 19 feet from the nearest adjacent property. For this conceptual design, it was assumed a variance could be obtained to reduce the property line setbacks to 10 feet. Without this variance, reconstructing the JBPS on the existing site is not feasible.

Facility Description

The new pump station on the existing site would include the following major design elements:

- A new wetwell developed from the existing caisson;
- A new drywell constructed adjacent to the existing caisson with four pumps rated for 4,200 gpm at 300 feet TDH;
- A new superstructure to house a 1,500 kW generator and electrical and control equipment; and
- A temporary 15.0 mgd peak capacity pump station, valving structure, and flowmeter vault.

Planning level design drawings are included in Appendix A.

To convey a peak flow of 24 mgd without surcharging, a new 36-inch diameter influent sewer would be required from the pump station to the first upstream manhole. Replacing the influent sewer with a larger diameter pipe was evaluated to increase the storage capacity of the upstream conveyance system. Given the short length of pipe (95 feet) that would be replaced, there is only a slight increase in detention time associated with an increased pipe size (Table 4-1). At this time, a 48-inch sewer was used which, if installed at a constant 0.5 percent slope, would have a full pipe capacity of 67 mgd.

Table 4-1. Influent Sewer Storage Volume

Pipe Diameter (in)	Storage (gal)	Detention Time at 4.5 mgd (min)
36	5,000	1.6
48	8,900	2.9
60	14,000	4.6
72	20,000	6.5
Notes: The 1997 ADWF was approximately 4.5 mgd (3125 gpm).		

Construction

Construction of this facility would be technically difficult. It was assumed that the temporary pump station could be located in Juanita Bay Park, just east of the vacated 93rd Avenue NE right-of-way and south of Juanita Drive. Both the park and the vacated right-of-way are owned by the King County Parks Department. The temporary pump station would need to operate throughout construction of the new pump station, which would take 1-1/2 to 2 years. Other construction concerns include the close proximity of residences to the pump station site. Construction noise and vibrations could generate a number of local resident complaints, since there is an apartment building approximately 10 feet from the western property line.

Construction of a new drywell would involve excavation at least as deep as the existing caisson. Prior to excavation for the drywell, it would be necessary to stabilize the existing caisson since the earth pressure on it would be unequal when the soil is removed to construct the new drywell. For this planning level evaluation, it was assumed driven steel sheet piling could be used to construct the below grade sections of the new pump station.

A simplified sequence of construction is as follows:

- Construct a temporary pump station, valving structure, and associated equipment (1 year).
- Connect temporary pump station to the existing force mains, and start-up pump station (2 months).
- Demolish the existing pump station superstructure and stabilize the caisson (3 months).
- Construct new pump station and influent sewer (1-1/2 to 2 years).
- Divert flow to the new pump station and maintain operation of the temporary pump station through start-up (2 months).
- Demolish the temporary pump station and abandon sections of the existing force mains (1 month).

Based upon this sequence, the construction of a new pump station on the existing site would take 3 to 4 years.

ALTERNATIVE 2 – NEW PUMP STATION ON A NEARBY SITE

Topography and land use limit the number of sites available for a new pump station. Most of the property surrounding the existing pump station is occupied by residential multifamily units. The King County Park Department owns a 150-foot by 165-foot lot across 93rd Avenue NE from the existing pump station that currently serves as a vehicle shop. The Juanita Bay Park, owned by King County, is diagonally across the street from the existing pump station. This 30-acre park is primarily located between Juanita Drive NE and Lake Washington and includes a 10-acre segment north of Juanita Drive NE and west of 97th Avenue NE.

The closest suitable site for a new pump station is the parcel across 93rd Avenue NE from the existing pump station. The close proximity of this site to the existing pump station site makes it less expensive to construct the influent sewer. Other possible sites for a new pump station are in Juanita Bay Park. For ease of access and to meet requisite setbacks from Juanita Creek and Lake Washington, the most practical sites are the parking lot just south of the intersection of Juanita Drive NE and 97th Avenue NE and the northwest corner of this intersection.

These sites are located approximately 1,300 feet from the existing pump station and are shown on Figure 4-1. Therefore, a slightly deeper influent sewer and pump station would be required. A 48-inch diameter influent sewer at a slope of 0.2 percent would have full pipe capacity of approximately 45 mgd and the influent sewer would provide an additional 39 minutes of storage at 4.5 mgd (1997 ADWF). The resulting invert elevation would be 88.5 feet (King County/Metro Datum), so the bottom of a pump station in this location would need to be approximately 2.5 feet lower than one at the site across the street from the existing pump station.

The potential to increase the size of the microtunnel to provide storage and thereby decrease the capacity of a new pump station located at the 97th Avenue NE site can be evaluated further at the time of predesign construction if this alternative is further developed.

Facility Description

The King County Parks site across the street from the existing pump station is significantly larger than the existing pump station site. As a result, the new pump station could include the following items not feasible as part of a new pump station on the existing site:

- Pig launches for each force main,
- Flowmeters located inside the pump station instead of in a vault, and
- More space around equipment for operations and maintenance.

Planning level drawings are included in Appendix A.

Construction

Construction of a new pump station on a new site would be less complex than trying to reuse the existing site for a new pump station. A temporary pumping station would not be required and the geotechnical challenges associated with bracing the existing caisson and excavating a new, adjacent drywell would be eliminated. The sequence of construction would generally be as follows:

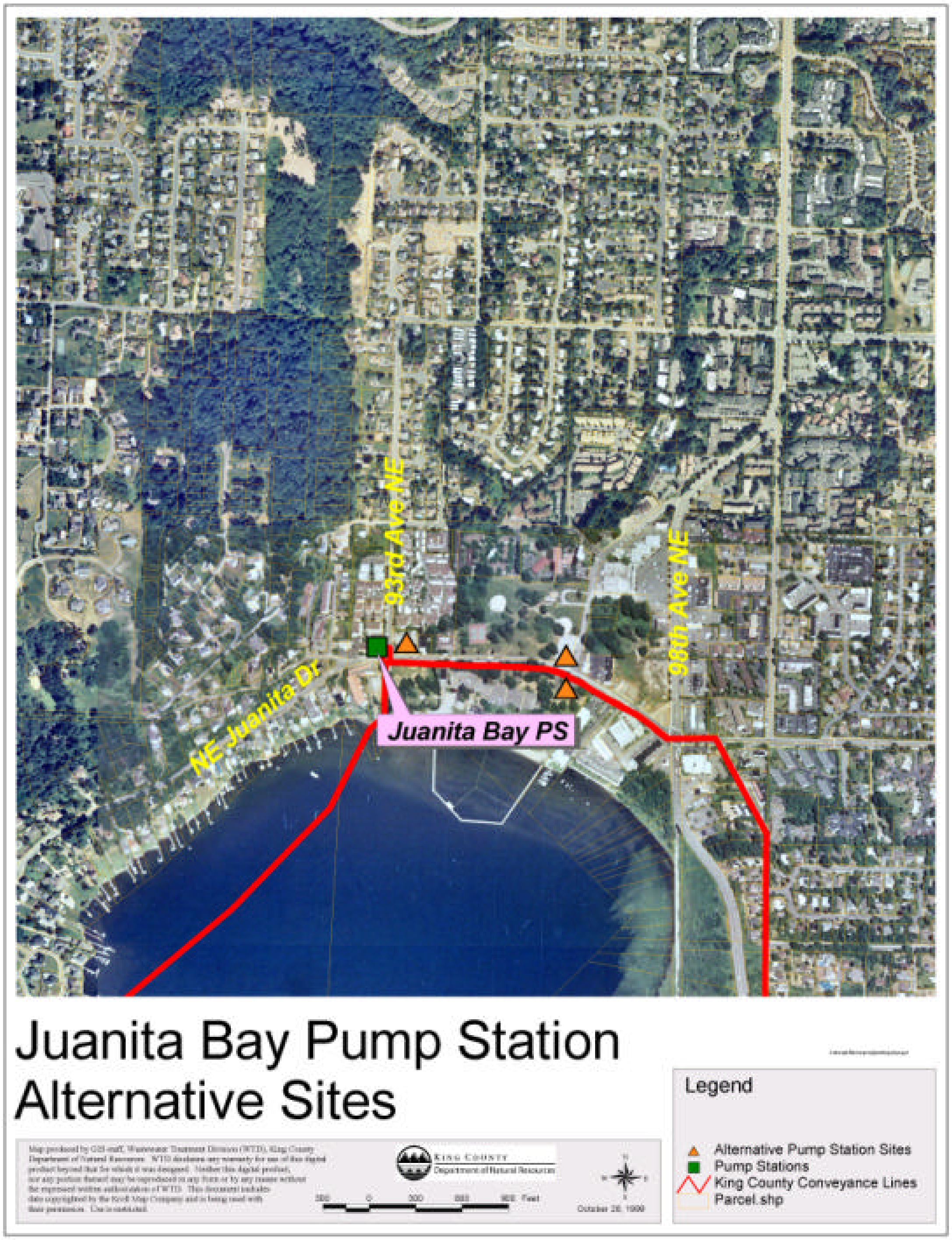


Figure 4-1: Pump Station Alternative Sites

- Construct new pump station and influent sewer (1-1/2 to 2 years).
- Divert flow to the new pump station and maintain operation of the existing pump station through start-up (2 months).
- Demolish existing pump station and abandon sections of unused force mains (2 months).

The total estimated time for construction is approximately 2-1/2 years, 1 to 2 years shorter than construction of a new pump station on the existing site.

ALTERNATIVE 3 – CONSTRUCT A NEW JUANITA HEIGHTS PUMP STATION

Construction of a new Juanita Heights Pump Station was evaluated to decrease the discharge head on the pumps at the JBPS. If a new Juanita Heights Pump Station were constructed, the static head on the pumps at the JBPS would decrease from approximately 163 feet to 45 feet and the total dynamic head at 24 mgd would decrease from approximately 300 feet to 150 feet. The pumps at the JBPS would have approximately 300 HP motors instead of the 500 HP motors required if only a new JBPS is constructed. However, the JBPS would still be required to pump the projected peak flow and the wetwell of the existing pump station would still have the same hydraulic problems. The variable frequency drives (VFDs) and generator would still need to be increased in size. Although decreasing the discharge head on the pump would decrease the size of the drivers, reconstruction of the pump station would still be required to convey the projected 20-year storm flow at build-out.

Previously, when the Juanita Connection included the Juanita Heights Pump Station, the system suffered from a number of operational problems including:

- The tendency of the pump controls at the Juanita Heights Pump Station to overshoot the required capacity following changes in flow thereby causing surging in the force main;
- The lack of stable speed regulation once the pump station was operating at or near the required speed; and
- The potential for a power failure to upset control of the Juanita Heights Pump Station to such a degree that the pump station would not be able to reliably restart automatically.[7]

These operational problems, combined with the increased costs of operating two pump stations, lack of reliability of the two pump station arrangement, institutional concerns, and the increased potential for overflows provided a strong case for the abandonment of the Juanita Heights Pump Station. For these same reasons, the reconstruction of the Juanita Heights Pump Station was not determined to be practical and the alternative was not evaluated further.

[7] Metro Juanita Connection System Predesign Study, Kramer, Chin and Mayo, Inc. January, 1982.

ALTERNATIVE 4 – UPGRADE OF EXISTING PUMP STATION AND STORAGE

The following improvements would be required for the pump station upgrade. These improvements are based on pump station problems noted in the previous chapter, County pump station design requirements, and applicable codes.

1. Standby Power: An emergency diesel generator sized to provide power to all the raw sewage pumps and other essential equipment including level and instrument control, HVAC equipment, sump pumps, water system pumps, and lighting. The standby power system would include a diesel fuel storage tank with sufficient storage capacity to operate the generator for 24 hours at full load.

2. Chemical Handling Facilities: The December 1998 draft of the *East Side Interceptor Chemical Injection Facility Feasibility Study* recommended adding 395 gallons of 25-percent caustic (NaOH) twice a week to the pump station wetwell to control sulfide generation in the JBPS force mains. A 2,500-gallon storage tank would be sufficient to provide three weeks of caustic storage at the pump station at this slug-dosing rate.

3. HVAC: Complete separation of the wetwell and drywell HVAC systems. Forced ventilation with at least 12 air changes per hour for the wetwell and at least 6 air changes per hour for the drywell.

4. Sound Attenuation: To protect the hearing of operation and maintenance personnel and avoid the need for a noise monitoring program, the sound level in the pump stations should be maintained at less than 85 dBA. Sound emanating from the station will also need to meet the maximum permissible environmental noise levels (WAC 173-60).

5. Wetwell Level Control: Backup floats for wetwell level control in the event the instrument air system fails.

Pump Station Upgrade

Space and other constraints limit the modifications that can be made within the existing pump station. These limitations include the following:

- The maximum size of the pump discharge piping is 12 inches.
- Pump suction pipes are cast beneath the pump room and wetwell floor slab and cannot be readily be modified while the pump station remains in service.
- The wetwell cannot be enlarged or significantly modified.
- Only one sump pump can be installed in the existing sump.

Additionally, as a result of the limited space within the pump station, a generator large enough to power all four pumps and chemical feed facilities would have to be located offsite or the existing superstructure rebuilt. Since construction of a new structure to house the generator and chemical feed facilities would be less expensive and less disruptive then demolition of the existing superstructure, construction of a new facility across 93rd Avenue Northeast from the pump station was assumed for planning purposes. This property is currently owned by the King County Parks Department and includes a vehicle shop on part of the site.

Given these site constraints and baseline criteria, alternatives were developed to maximize the utilization of the existing pump station and compare the operation of the pump station at higher flows to commonly used design parameters.

Alternatives for Increasing Pump Station Capacity

The flow through the pump station was compared to a number of hydraulic parameters in an effort to determine the flows at which certain hydraulic criteria are exceeded. These hydraulic parameters included the following[8,9].

- Maximum horizontal velocity in the wetwell: 1.0 feet-per-second.
- Maximum velocity in the suction inlet: 3.5 feet-per-second.
- Maximum velocity in the suction piping: 5.0 to 8.0 feet-per-second.
- Maximum velocity in the discharge piping: 8.0 to 12.0 feet-per-second.

Wetwell and Pump Suction

Horizontal velocities in the wetwell are currently higher than 1.0 fps, a commonly used design parameter for wastewater wetwells, at peak flows (Table 4-2). The pump station was designed with vortex suppressor plates in the draft tube inlets. However, the vortex suppressors in the pump station may have actually created vortexes.[10] In addition, the velocities in the draft tubes at the pump inlet currently exceed commonly accepted design parameters (Table 4-3). If flows are increased, the pump suction velocities will only increase further beyond the recommended maximum velocity.

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[8] Pumping Station Design, Sanks, et al., 1st Edition. (1989). Pg. 312.

[9] Pumping Station Design, Sanks, et al., 2nd Edition. (1998). Pg. 341, 361.

[10] Recommended design includes holes in the vortex suppressors. "The holes in (the vortex suppressor) must be adequate for distributing any eddies shed at the free edge of the plate by cross flow to prevent the plate itself from becoming a vortex generator" (Ref. Pumping Station Design, Sanks et al, pg. 310).

Table 4-2. Horizontal Velocities in the Wetwell

$Q_{pump\ station}$ (mgd)	V_{horiz} [1] (fps)	Notes
7.1	1.0	Maximum recommended velocity.[2]
14.2	**2.0**	At nominal peak capacity of the pump station.
28.0	**4.0**	Projected peak flow 2050.

Notes:
1. Horizontal velocity calculated in channel just upstream of the inlet to the first pump. Velocities in excess of the recommended value are highlighted.
2. Reference: Pump Station Design, Sanks et al, 1st Ed. (1989). Pg. 312.

Table 4-3. Pump Suction Velocities

$Q_{pump\ station}$ [1] (mgd)	Q_{pump} [1] (mgd)	$V_{draft\ tube\ inlet}$ (fps)	$V_{draft\ tube\ at\ pump\ inlet}$ [2] (fps)	Notes
8.5	2.6	1.3	5.0	Maximum recommended velocity in pump suction.[3]
13.6	4.1	2.0	8.0	Maximum recommended velocity in pump suction.[4]
14.2	4.3	2.1	**8.3**	At nominal peak capacity of the pump station.
23.7	7.1	3.5	**13.9**	Maximum recommended velocity in draft tube inlet.[5]
24.0	7.2	**3.5**	**14.1**	Future pump discharge based on 24-mgd peak pumping capacity for the station.

Notes:
1. Assuming the two force mains are not tied together and all the pumps are the same size, then there will be approximately a 40/60 flow split between the two force mains. Since all the pump suction lines are the same size, Q_{pump} and $Q_{pump\ station}$ represent the more restrictive case where the suction line carries 30 percent of the pump station flow.
2. Calculated at narrowest point in draft tube just upstream of the pump.
3. Reference: Pump Station Design, Sanks et al, 1st Ed. (1989). Pg. 312.
4. Reference: Pump Station Design, Sanks et al, 2nd Ed. (1998). Pg. 361.
5. Reference: Pump Station Design, Sanks et al, 1st Ed. (1989). Pg. 312.
6. Velocities in excess of the lowest recommended parameter are highlighted.

The wastewater velocities in the wetwell currently exceed commonly used design parameters. An increase in these already high velocities can reasonably be expected to lead to an increase in surface and subsurface vortices that adversely affect pump performance. A physical or numerical hydraulic model of the pump station wetwell should be developed and tested to determine the maximum hydraulic capacity of the wetwell as part of evaluating increasing the capacity of the pump station. This model would provide a more comprehensive analysis of the wetwell hydraulics than can be obtained from comparison of the wetwell hydraulics to commonly used design parameters.

Pump Discharge Piping

As noted earlier, the 12-inch diameter pump discharge lines cannot be replaced since there is limited space within the pump room. These 12-inch diameter discharge lines connect to a 12-inch diameter header and an 18-inch diameter header in the motor room. It is possible to replace the existing 12-inch discharge header with an 18-inch discharge header. However, there is not sufficient space to replace the existing 18-inch header with a larger pipe. Given these constraints and assuming the force mains are operated independently in the same manner as they are now, were compared the velocities through the pump discharge lines to commonly used design parameters for a range of predicted flow conditions (Tables 4-4 and 4-5).

Table 4-4. Pump Discharge Line Velocities

$Q_{pump\ station}$ [1] (mgd)	Q_{pump} [1] (mgd)	V_{pump} [1] (fps)	Notes
13.5	4.1	8.0	Maximum recommended velocity, Reference 2.
14.2	4.3	**8.4**	At nominal peak capacity of the pump station.
20.3	6.1	**12.0**	Maximum recommended velocity, Reference 3.
28.0	8.4	**16.5**	Projected peak flow 2050.

Notes:
1. Assuming the two force mains are not tied together and all the pumps are the same size, then there will be approximately a 40/60 flow split between the two force mains. Since all the pump discharge lines are the same size, , Q_{pump}, $Q_{pump\ station}$, and V_{pump} represent the more restrictive case where the discharge line carries 30 percent of the pump station flow.
2. Reference: Pump Station Design, Sanks et al, 1st Ed. (1989). Pg. 312.
3. Reference: Pump Station Design, Sanks et al, 2nd Ed. (1998). Pg. 361.
4. Velocities in excess of the lowest recommended parameter are highlighted.

Table 4-5. Pump Discharge Header Velocities

$Q_{pump\ station}$ [1] (mgd)	Q_{header} (mgd)	V_{header} [1] (fps)	Notes
14.2	8.5	7.5	At nominal peak capacity of the pump station.
15.2	9.1	8.0	Maximum recommended velocity, Reference 2.
22.8	13.7	**12.0**	Maximum recommended velocity, Reference 3.
28.0	16.8	**14.8**	Projected peak flow 2050.

Notes:
1. Assuming the two force mains are not tied together and all the pumps are the same size, then there will be approximately a 40/60 flow split between the two force mains. Under these conditions, the velocity in the 18-inch header connected to the 24-inch force main will experience higher peak flows.
2. Reference: Pump Station Design, Sanks et al, 1st Ed. (1989). Pg. 312.
3. Reference: Pump Station Design, Sanks et al, 2nd Ed. (1998). Pg. 361.
4. Velocities in excess of the lowest recommended parameter are highlighted.

Summary of Hydraulic Criteria

The hydraulic criteria for the wetwell, pump suction, and pump discharge will exceed commonly referenced criteria at various pump station flowrates. The flows at which these criteria are exceeded are listed to help clarify which criteria have been exceeded at a given pump station discharge (Table 4-6). Although hydraulic modeling may determine that the capacity of the wetwell can be increased above the current 14.2 mgd peak pump station capacity, other factors may limit the peak pump station capacity. At 15.2 mgd, the flow through both the pump discharge lines and headers will exceed 8 feet per second. At approximately 19 to 20 mgd, the capacity of the influent sewer will be exceeded. Based on these hydraulic criteria and constraints, it is doubtful that the existing pump station can convey flows greater than 15 mgd and even less likely that the pump station can convey 19 mgd.

Table 4-6. Summary of Design Criteria Exceeded

$Q_{\text{pump station}}$ (mgd)	Criteria Exceeded
7.1	Horizontal velocity in wetwell exceeds 1.0 fps.
8.5	Pump suction velocity exceeds 5.0 fps.
13.5	Pump discharge velocity exceeds 8.0 fps.
13.6	Pump suction velocity exceeds 8.0 fps.
15.2	Discharge header velocity exceeds 8.0 fps.
19-20	The influent sewer surcharges.
20.3	Pump discharge velocity exceeds 12.0 fps.
22.8	Discharge header velocity exceeds 12.0 fps.

For any increase in the peak capacity of the pump station, the pumps, motors and VFDs would need to be replaced since the higher headloss though the discharge piping and force mains would increase the motor horsepower requirements. An increase in flow to 19 mgd would cause the wetwell to operate with a horizontal velocity of 2.7 feet per second and pump suction velocity of 11.3 feet per second, well above normal design parameters.

If the capacity of the pump station is increased, there are other improvements to the pump station that should be performed simultaneously to ensure that the pump station functions well and is up to King County standards and applicable codes. These improvements include a new building to house a new standby generator, chemical feed facilities, and other improvements previously outlined.

Limitations of Upgrade

There are a number of limitations of any upgrade of the JBPS. These limitations include:

- **There is only one sump pump in the pump station.** The sump is not large enough to accommodate a second pump. Placement of a second sump pump on the floor of the pump room would constitute a tripping hazard and would make access to the raw sewage pumps more difficult.

- **Equipment access is limited.** Most notably access is limited to the seal water control valves and sump pump, which are both located behind Raw Sewage Pump Number 104. There is also a limited amount of clearance around the pumps. Good design practice includes providing 42 inches on three sides of all major equipment. In the case of the raw sewage pumps, only two sides of the pump can be readily accessed. Of the two sides that are accessible, access on one side is limited to only 24 inches by the piping for the adjacent pump.

- **The spiral stairs would have to remain.** Although spiral stairs are acceptable for certain limited situations described in the UBC and OSHA regulations, they are not recommend for new facilities when other means of egress can be provided[11,12]. However, the spiral stairs are already part of the pump station and replacement of the spiral stairs in the existing pump station with another means of egress is not feasible given the space constraints of the existing facility.

In addition, it does not appear feasible to convey peak flows at build-out through the existing pump station. Therefore, eventually either a new pump station will have to be constructed, or a significant amount of upstream storage would be needed to attenuate the peak flows to the pump station.

Upstream Storage

The 20-year peak flow hydrograph was developed for the Juanita Bay Pump Station for the year 2050 20-year peak flow of 28.0 mgd to determine the requisite volume of storage for two alternatives. These two alternatives include: (1) maintaining the existing pump station peak capacity of approximately 14 mgd, and (2) increasing the pump station capacity to 19 mgd. The required storage volume was estimated by calculating the area under the peak flow curve and above the peak pump station capacity as shown on Figure 4-1. Based on this approach, it was estimated that 6.6 million gallons (MG) of storage would be required to reduce the peak flow from 28.0 mgd to a peak pump station capacity of 14 mgd. If the peak capacity of the pump station could be increased to 19 mgd, then approximately 1.7 MG of storage would be required. Costs were developed based on these storage requirements and required upgrades of the existing pump station.

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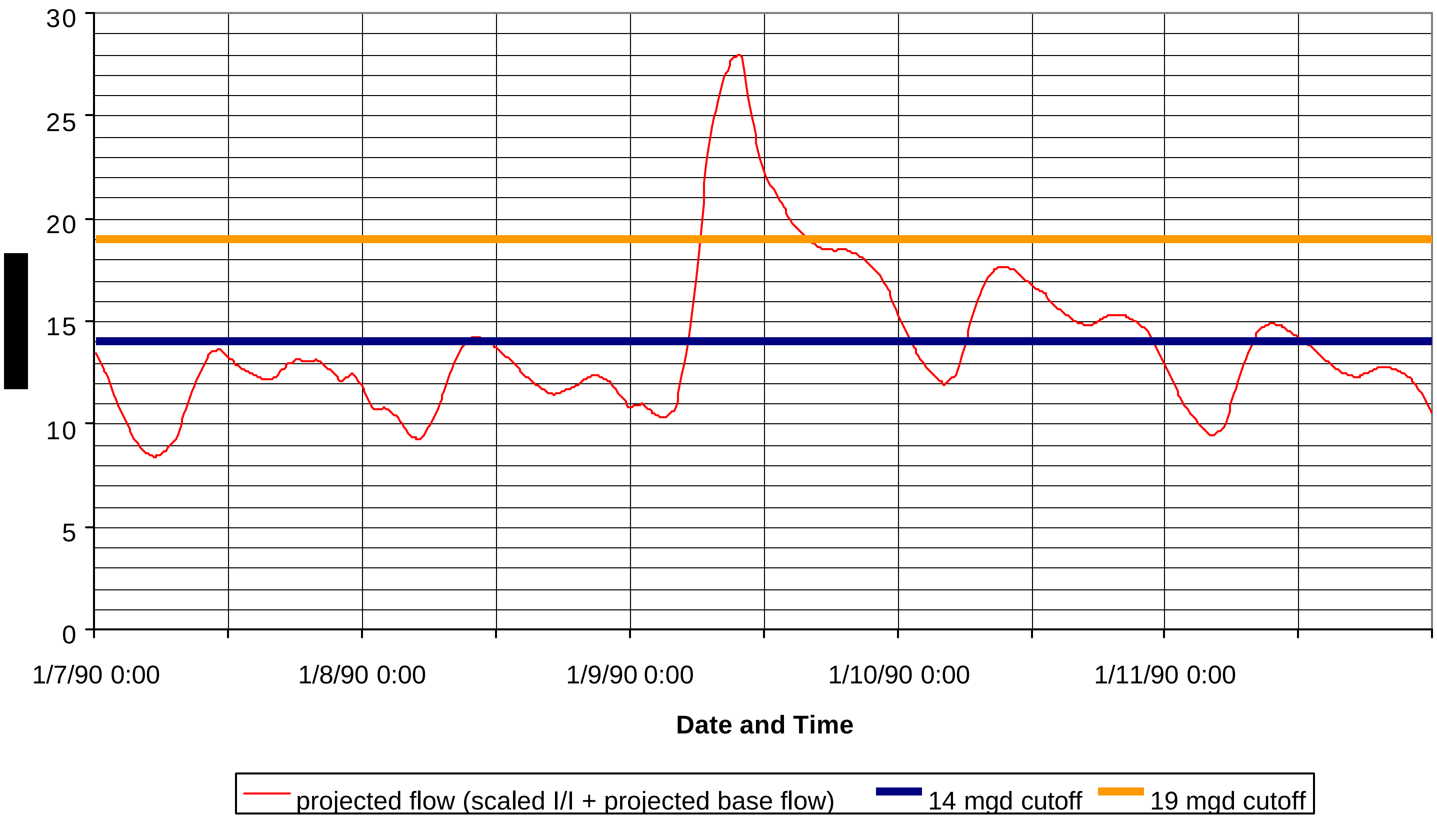
[11] Uniform Building Code (1997 – Latest Edition)- Section 1003.3.3 –"Stairs or ladders used only to attend equipment or window wells are exempt from the requirements of this section."

[12] OSHA 1910.24 (b) – " . . .Spiral stairs shall not be permitted except for special limited usage and secondary access situations where it is not practical to provide a conventional stairway."

FIGURE 4-2
JUANITA BAY PUMP STATION PEAK FLOW HYDROGRAPH
AND STORAGE ESTIMATE



Construction

Construction of a storage facility and upgrading the existing pump station could be performed independently of each other. As a result, the duration of construction would probably be shorter than any of the other alternatives. The sequence of construction would be relatively straightforward:

- Upgrade the existing pump station and construct a new peak flow storage facility (1-1/2 to 2 years).
- Start-up and test the pump station and storage facility (2 months).

The total estimated time for construction is approximately 1-1/2 to 2 years, 6 month shorter than construction of a new pump station.

COST ESTIMATES

Capital costs developed for each alternative are summarized in Table 4-7. These costs reflect costs for facilities to convey a maximum of 24.0 mgd, slightly greater than the projected 20-year peak flow in 2050 with no increase in I/I over 1990 rates. Based on these planning level cost estimates, the least expensive options to convey peak flows at build-out are (1) to construct a new pump station on the site across 93rd Avenue NE from the existing pump station, and (2) to upgrade the existing pump station and provide 1.5 MG of upstream storage. The construction of a temporary pump station to convey flows significantly increases the cost of construction of a new pump station on the existing site. The only other sites that may be feasible for a new pump station are located approximately 1,300 feet from the existing station. The influent sewer to this new station would need to be microtunneled this distance at a depth of approximately 35 feet. This microtunnel is necessitated not only by the depth, but because the influent sewer would cross under Juanita Creek, a Class A stream. This microtunnel, access shafts, and the increased depth of the pump station would increases the total construction cost by approximately $3.0 million.

Table 4-7. Estimated Pump Station Capital Costs (1999 Dollars)

Alternative	Total Construction Cost (Millions)	Total Estimated Capital Cost (Millions)[1]
1 - New Pump Station on the Existing Site	$8.8	$11.9
1 - Temporary Pump Station	$3.9	$5.3
1 - Total	**$12.7**	**$17.2**
2A - New Pump Station on 93rd Avenue NE Site	$10.9	$15.3
2B - New Pump Station on 97th Avenue NE Site	$13.9	$19.4
3 - Reconstruct Juanita Heights Pump Station	Not Recommended	Not Recommended
4A – Upgrade the existing pump station with no capacity increase and construct a new generator and chemical building	$2.7	$4.3
4A – 6.6 million gallons of storage[2]	$28.4	$38.3
4A – Total	**$31.1**	**$42.6**
4B – Upgrade the existing pump station to 19.0 mgd peak capacity and construct a new generator and chemical building	$3.2	$5.0
4B – 1.7 million gallons of storage[2]	$7.3	$9.9
4B – Total	**$10.5**	**$14.9**

Notes:
(1) Total estimated capital cost includes 35 percent of the total direct project cost for engineering, administration, permits, land acquisition, easements, and fees.
(2) Construction cost for storage based on $4.30 per gallon.

The costs to upgrade the pump station and provide storage include a new generator, electrical, and chemical feed facility on the King County Parks vehicle shop site across 93rd Avenue Northeast from the JBPS and improvements to the raw sewage pumping and ancillary systems in the existing pump station.

These planning level costs were developed based on previous detailed cost estimates for recently constructed King County pump stations and standard cost criteria. These cost criteria include:

- A construction cost contingency of 30 percent,
- Sales tax of 8.6 percent of the estimated construction cost, and
- Allied costs of 35 percent of the total estimated direct project cost.

The allied cost factor is considered to cover King County facility management, consulting services, and insurance. Land acquisition costs were estimated to be $25 per square foot, and $5 per square foot for easements. Detailed planning level cost estimates are included in Appendix B.

The estimated annual operation and maintenance costs for each alternative was $187,000 with a net present value of $1.8 million for a 20-year period. These costs were equal for each alternative since the total flow conveyed for each alternative would be the same. Hence, these operation and maintenance costs were not used to evaluate relative costs between the alternatives.

In summary, the cost to construct a new pump station and upgrade the existing pump station to 19.0 mgd and construct 1.5 MG of storage are approximately equal in cost. In addition, constructing a new station or upstream storage would take approximately 1 to 2 years less to construct than a new pump station on the existing JBPS site, and would not require a variance from setback requirements.

Chapter 5 – Conclusions and Recommendations

The peak capacity of the existing Juanita Bay Pump Station, 14.2 mgd, is well below the projected 20-year peak flow to the pump station of 19.9 mgd in 2000. Both peak and base flows are projected to increase to the pump station as the population served grows and the collection system deteriorates with age. Even if the projected deterioration within the basin does not occur, or an I/I program is implemented that can offset deterioration of the system, the peak 20-year storm flow is projected to reach 20.2 mgd in 2010 and 23.4 mgd in 2050. These projected flows are well above the capacity of the existing pump station.

Comparison of Alternatives

Based on these flow projections, five alternatives were evaluated to either upgrade or replace the existing pump station. These five alternative are:

1. A new pump station on existing site;
2. A new pump station on the King County Parks parcel at the corner of Juanita Drive NE and 93rd Avenue NE across the street from the existing pump station;
3. A new pump station in Juanita Beach Park at the corner of Juanita Drive NE and 97th Avenue NE;
4. A new Juanita Heights Pump Station; or
5. An upgrade of the existing pump station to 19 mgd combined with storage upstream of the pump station.

These alternatives are compared in Table 5-1, with the exception of the Juanita Heights Pump Station, which was dropped from further consideration for the reasons outlined in the previous chapter. In addition, constructing 6.6 MG of storage, the volume required if the capacity of the pump station is not increased, was dropped from further consideration due to the high cost of this alternative.

Based on the comparison of the alternative in Table 5-1, constructing a new pump station on the existing pump station site was dropped from further consideration because the local impacts and construction constraints are so great that it appears impractical to use the site for a new pump station. Therefore, the three most practical alternatives are

Table 5-1. Comparison of Alternatives

Alternative	Operations and Maintenance	Environmental/ Permitting	Right of Way/ Easements	Geotechnical Issues	Local Impacts and Construction Constraints	Duration of Construction	Total Capital Cost (Millions)
Pump Station on Existing Site	Facility is a force fit to the existing site. Potential space constraints.	Would require exemption from City of Kirkland setback and landscaping requirements.	Easement required for temporary pump station and valve structure in the Juanita Beach Park.	Existing caisson would need to be stabilized due to unequal loading during and after construction.	Construction of new pump station severely constrained by work required adjacent to the existing pump station. Pump station would be within 20 feet of adjacent apartments. Inadequate space for construction staging.	3 – 4 years	$17.2
Pump Station at 93rd Avenue NE	Designed to latest KC WTD standards with adequate access to equipment.	Exemptions from City of Kirkland land use regulations not required.	Purchase of 150'x165' parcel required.	Caisson as recommended. High groundwater.	No major impacts or constraints. Pump station over 60 feet from the nearest building.	2 - 2.5 years	$15.3
Pump Station at 97th Avenue NE	Designed to latest KC WTD standards with adequate access to equipment.	Would require tunneling underneath Juanita Creek.	Purchase of parcel within Juanita Beach Park required. Easements required for microtunnel to the pump station.	Caisson as recommended. High groundwater.	Pump station would need to be integrated with the plan for Juanita Beach Park.	2 - 2.5 years	$19.4
Upgrade the Existing Pump Station and Storage	Operation, maintenance, and access problems with the existing pump station.	Permitting required for the storage facility. Must be setback more than 75’ from Juanita Creek and 200’ from Lake Washington.	Purchase of 150'x165' parcel required. Easements required for storage facility.	Soft ground tunneling required if storage placed in Juanita Beach Park. High groundwater.	A suitable storage site or sites would need to be located upstream of the pump station. Local impacts would be no greater than constructing a new pump station.	1.5 - 2 years	$14.9

- A new pump station at the corner of Juanita Drive NE and 93rd Avenue;
- A new pump station at the corner of Juanita Drive NE and 97th Avenue NE; and
- An upgrade of the existing pump station combined with storage upstream of the pump station.

RECOMMENDED PROJECT

At this time, the most practical alternative appears to be construction of a new 24-mgd peak capacity pump station expandable to a maximum peak capacity of 28 mgd at the corner of 93rd Avenue NE and Juanita Drive NE. This alternative appears to be the most practical for the following reasons:

Operations and Maintenance – The pump station would be designed in accordance with the latest King County standards for pump stations, provide adequate equipment access, and provide a more reliable pump station than the existing facility to minimize the possibility of overflows due to mechanical failure.

Environmental and Permitting – The facility would be located north of Juanita Drive NE, well over 200 feet from the shore of Lake Washington and more than 75 feet from Juanita Creek, minimizing the number of permits required. Since the parcel is currently zoned park, the setbacks and landscaping requirements are determined on a case-by-case basis. However, the parcels is large enough so the pump station could be located with more than a 20-foot setback, as required for the existing pump station parcel, and comply with the most stringent landscaping requirements. A request for an exemption or variance from the City of Kirkland land use regulations would most likely not be required, unlike construction of a replacement pump station on the existing site.

Right of Way and Easements – The 150-foot by 165-foot parcel currently owned by the King County Parks Department would need to be purchased and an easement would be required for a portion of the vacated 93rd Avenue NE right-of-way owned by the King County Parks Department. However, construction of a pump station in the vicinity of 97th Avenue NE and construction of a chemical feed and generator facility as part of an upgrade of the existing pump station would also require purchasing the same parcel from the King County Parks Department.

Geotechnical Issues – Based on soils investigation for the existing pump station, a caisson method of construction was determined to be the most feasible and was therefore recommended. The caisson method of construction could be employed for a new pump station but not for a new pump station on the existing site where driven steel sheet piling, tied-back solider piles, or other applicable excavation support would be required.

Construction Constraints – A pump station on the 150-foot by 165-foot parcel could be located over 65 feet from the nearest structure, minimizing the impacts of construction on adjacent residences. In addition, this parcel has adequate space for construction staging unlike the existing pump station site.

Total Project Costs – The estimated total project cost of the recommend alternative is \$15.3 million, more than \$4 million less than a new pump station near the corner of 97th Avenue and Juanita Drive.

Therefore, constructing a new pump station on the site across 93rd Avenue NE from the existing pump station appears to be the least cost, most feasible, and easiest to permit and construct facility of the alternatives evaluated.

I/I control should be implemented for the basin since a 28-mgd peak flow would necessitate replacing the existing 18-inch force main since the velocities in these force mains would be higher than a practical maximum velocity is 8 fps. At 28.0 mgd, the velocity in the 24-inch force main would be 9.4 fps and 7.8 fps in the 18-inch force main assuming equal headloss between the two force mains. Therefore, unless an I/I program is implemented or deterioration of the collection system is not as great as projected, replacement of the force mains will be warranted for hydraulic reasons by 2016.

A preliminary list of the permits required for the recommended pump station include the following:

- Building Permit (City of Kirkland);
- Grading Permit (City of Kirkland);
- State Environmental Policy Act (SEPA) Checklist since the gross square footage of the project would probably exceed 4,000 square feet; and
- Phase IIA Permit (City of Kirkland) which is similar to a Conditional Use Permit.

Projected 20-year peak flows currently exceed the peak capacity of the existing pump station. Therefore, planning, predesign, and associated work on this pump station should begin shortly. A preliminary schedule for the predesign, design, permitting, construction and associated work indicates that it would probably take almost 5 years to permit, design, and construct a new Juanita Bay Pump Station as shown on Table 5-2.

Table 5.2: Preliminary Construction Schedule

Task	Schedule
Predesign, Property Acquisition, Geotechnical	January, 2000 - August, 2000
Permitting and Easements	April, 2000 - November, 2000
Final Design, Bidding	September, 2000 - January, 2002
Construction	February, 2002 - February, 2004
Start-Up	March, 2004 - May, 2004